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DESIGN OF STRUCTURES TO RESIST THE EFFECTS OF ATOMIC WEAPONS

SHEAR WALL STRUCTURES

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ENGINEERING AND DESIGN

DESIGN OF STRUCTURES TO RESIST THE EFFECTS OF ATOMIC WEAPONS

SHEAR WALL STRUCTURES

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ENGINEERING AND DESIGN

DESIGN OF STRUCTURES TO RESIST THE EFFECTS OF ATOMIC WEAPONS

SHEAR WALL STRUCTURES

INTRODUCTION

9-01 PURPOSE AND SCOPE. This manual is one in a series issued for the guidance of engineers engaged in the design of permanent type military structures required to resist the effects of atomic weapons. It is applicable to all Corps of Engineers activities and installations responsible for the design of military construction.

The material is based on the results of full-scale atomic tests and analytical studies. The problem of designing structures to resist the effects of atomic weapons is new and the methods of solution are still in the development stage. Continuing studies are in progress and supplemental material will be published as it is developed.

The methods and procedures were developed through the collaboration of many consultants and specialists. Much of the basic analytical work was done by the engineering firm of Ammann and Whitney, New York City, under contract with the Chief of Engineers. The Massachusetts Institute of Technology was responsible, under another contract with the Chief of Engineers, for the compilation of material and for the further study and development of design methods and procedures.

It is requested that any errors and deficiencies noted and any suggestions for improvement be transmitted to HQDA (DAEN-MCE-D) WASH DC 20314. *

9-02 REFERENCES. Manuals - Corps of Engineers - Engineering and Design, containing interrelated subject matter are listed as follows:

DESIGN OF STRUCTURES TO RESIST THE EFFECTS OF ATOMIC WEAPONS

EM 1110-345-413 Weapons Effects Data
EM 1110-345-414 Strength of Materials and Structural Elements
EM 1110-345-415 Principles of Dynamic Analysis and Design

EM 1110-345-416 Structural Elements Subjected to Dynamic Loads
EM 1110-345-417 Single-Story Frame Buildings
EM 1110-345-418 Multi-Story Frame Buildings
EM 1110-345-419 Shear Wall Structures
EM 1110-345-420 Arches and Domes
EM 1110-345-421 **Buried and Semiburied Structures**

a. References to Material in Other Manuals of This Series. In the text of this manual references are made to paragraphs, figures, equations, and tables in the other manuals of this series in accordance with the number designations as they appear in these manuals. The first part of the designation which precedes either a dash, or a decimal point, identifies a particular manual in the series as shown in the table following.

EM	paragraph	figure	equation	table
1110-345-413	3-	3.	(3.)	3.
1110-345-414	4-	4.	(4.)	4.
1110-345-415	5-	5.	(5.)	5.
1110-345-416	6-	6.	(6.)	6.
1110-345-417	7-	7.	(7.)	7.
1110-345-418	8-	8.	(8.)	8.
1110-345-419	9-	9.	(9.)	9.
1110-345-420	10-	10.	(10.)	10.
1110-345-421	11-	11.	(11.)	11.

b. Bibliography. A bibliography is given at the end of the text. Items in the bibliography are referenced in the text by numbers inclosed in brackets.

c. List of Symbols. Definitions of the symbols used throughout this manual series are given in lists following the table of contents in EM 1110-345-413, EM 1110-345-414, EM 1110-345-415, and EM 1110-345-416.

9-03 RESCISSIONS. Draft EM 1110-345-419 (Part XXIII - The Design of Structures to Resist the Effects of Atomic Weapons, Chapter 9 - Design of Shear Walls).

9-04 BEHAVIOR OF SHEAR WALL STRUCTURES. Shear wall structures respond to lateral loads in a somewhat different manner than rigid frame structures. The basic difference between the two types of construction is the manner in which lateral loads are transmitted to the foundation. In rigid frame structures the columns transmit the lateral forces mainly through bending of the columns, whereas in shear wall structures the shear walls transfer

the lateral loads to the foundation through both bending and shearing action of the shear walls. Shear walls are inherently strong and will resist large lateral forces. For this reason shear wall construction will normally result in a more economical structure for blast resistance than will rigid frame construction. This type of construction has been found to be the most advantageous for earthquake-resistant structures and is widely used in the United States and other countries.

Shear wall structures are very stiff compared with rigid frame structures. The recommended design procedure for blast loads is based on a limitation of the deflection to that at which the ultimate load is developed as defined in paragraph 4-13 of EM 1110-345-414. In shear wall structures, columns are usually provided between the shear walls to carry the vertical loads including blast loads on the roof. The columns in shear wall structures are not subject to the same degree of bending from the lateral loads as in rigid frame structures and can therefore be of smaller cross section than in a comparable rigid frame building designed for blast loads.

A blast load applied to the front wall of a shear wall structure, as shown in figure 9.1, is transmitted through the roof and floor slabs to the shear walls and thus to the foundation. The front wall of the structure spans vertically between the foundation, the floor, and the roof slab. The upper floor and roof slabs act as deep beams, and in turn transmit the front wall reactions to the shear walls. In most cases a small portion of the front wall load will be transmitted directly to the front edge of the shear wall but this may be ignored in the design procedure.

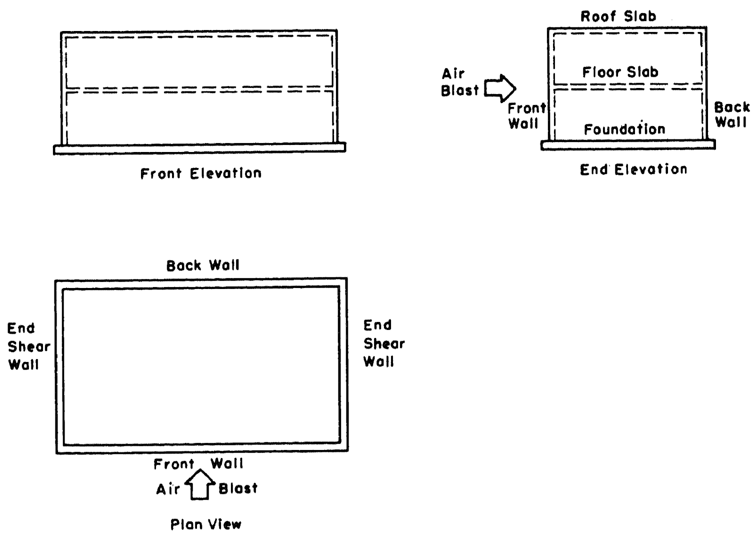


Figure 9.1. Simple shear wall structure

The roof and floor slabs act as the webs of I-shaped or channel-shaped

deep beams with portions of the front and rear walls acting as the flanges. In these deep beams the shear distribution is essentially uniform over the depth of the web. Hence the assumption is made that the horizontal roof and floor slab reactions are uniformly distributed along the shear walls. Horizontal blast loads applied to the back walls of the structure are carried in a similar manner through the roof and floor slabs to the shear walls. The floor and roof slabs are designed as deep beams to remain in the elastic range.

Shear walls are designed for plastic behavior and may be considered as vertical cantilever beams, supported at the base, and loaded with horizontal loads at each floor level. Vertical loads are caused by the blast forces on the roof slab. In addition, vertical shearing forces are developed along the front and back edges of each shear wall due to the unbalanced forces between the roof and foundation on both the front and back walls (fig. 9.2). The front and back walls act integrally with the shear wall in resisting bending. This interaction, however, will be reduced where the front and back walls are badly cracked. The effective width of the front and back walls which can be considered to act as flanges is approximately one-third of the shear wall height, each side of the web above the section being considered [1]. This conclusion is based upon theoretical investigations [2] to determine the effective width of a T-section for an ideal continuous beam, and using a beam analogy to apply the results to a cantilever wall. A flange width of one-sixth of the shear wall height each side of the web should be used if the front and back walls are designed for plastic behavior in vertical bending between floors.

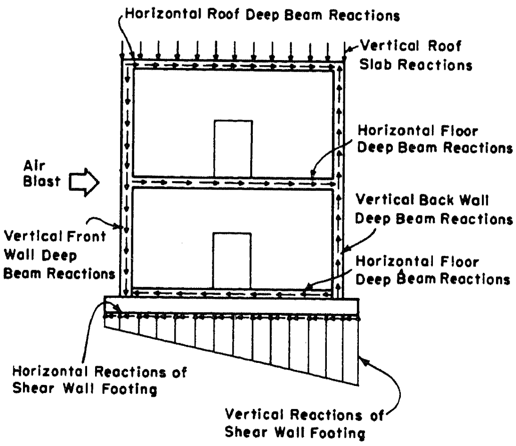


Figure 9.2. Forces acting on shear wall

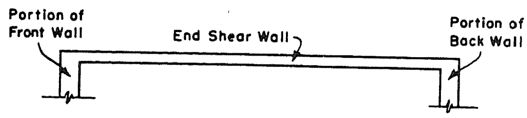


Figure 9.3. Shear wall section

A shear wall structure may have steel roof framing as illustrated

in figure 9.4. Steel roof trusses carry the vertical loads while horizontal trusses carry the lateral blast forces to the shear walls.

A structure with interior shear walls is illustrated in figure 9.5. Interior shear walls usually contain openings for corridors and doorways. The corridor walls may or may not be structural walls. The total stiffness or resistance to lateral deflection of a shear wall with openings is determined by a rigid frame analysis using both shear and moment deformations. The presence of integral corridor walls which may act as flanges should be taken into account.

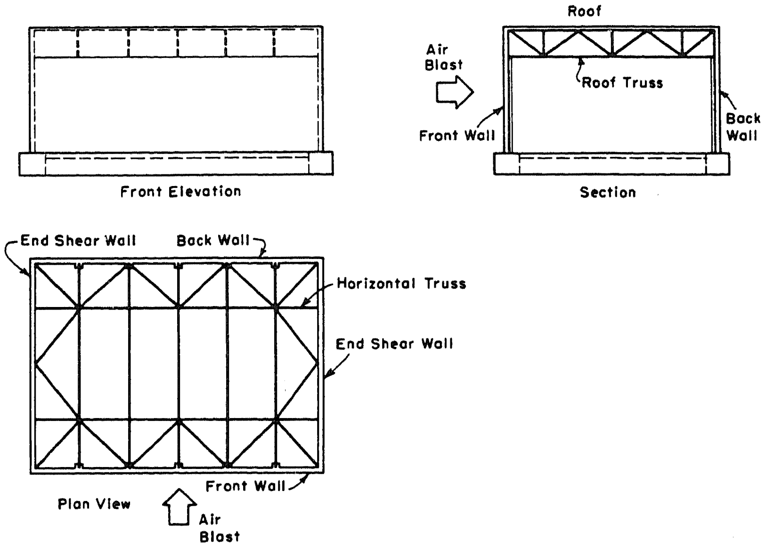


Figure 9.4. Shear wall structure with steel framing

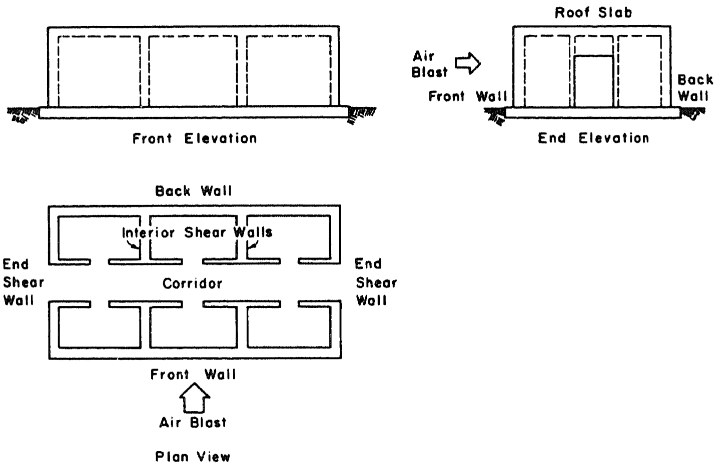


Figure 9.5. Shear wall structure with interior shear walls

The distribution of lateral loads to shear walls by roof and floor slabs acting as horizontal deep beams is an indeterminate problem dependent upon the stiffness of the floor slab and the relative stiffness of the walls. One approach is to assume that the roof and floor slabs are rigid plates and to divide the load among the shear walls in proportion to their relative stiffnesses. This is standard procedure in earthquake-resistant design [3, 4] and is a good approximation of the load distribution to the shear walls for lateral blast loads. This procedure implies equal deflection of all walls in the absence of torsion.

If the shear wall arrangement is not symmetrical, torsional stresses will be developed in the plane of the floor and roof slabs. These torsional stresses are distributed to all the walls which support the roof and floor and the more rigid walls will carry the large share of the torsion. It is recommended for this case that the assumption be made that the torsional stresses are carried by the walls which are normal to the shear walls because of the plastic deformations which the shear walls may undergo compared to the small elastic deformations of the other walls.

These assumptions must be applied with care and good engineering judgment. Cases will no doubt be encountered where the flexibility of the deep beam as well as the torsional stresses must be considered in the distribution of the lateral forces to the shear walls.

The moment and shear stresses in the roof and floor slabs resulting from deep beam action in transmitting the lateral blast loads to the shear walls can be determined by a moment distribution analysis. The shear wall reactions determined from this moment distribution solution may be used as a check of the shear wall reactions determined from the rigid plate assumption. The moment and shear stresses in the planes of the front and back walls can be determined through a moment distribution of the unbalanced vertical forces on these elements.

a. Design Criteria. The behavior of reinforced concrete shear walls subjected to lateral loads has been determined to some extent by test programs at Massachusetts Institute of Technology and Stanford University. See bibliography in EM 1110-345-414.

As discussed in paragraph 4-13 of EM 1110-345-414, the available data permits the determination of the shear resistance at which the wall first begins to crack. The corresponding elastic deflection is computed as a combination of bending deformation, using the moment of inertia of the gross concrete section with flanges, and shear deformation, using the gross shear wall web section. The ultimate shear resistance is determined as a function of the steel reinforcement in the shear wall panel and along the edge of the wall in compression. The deflection at which the ultimate shear load is developed is a function of the deflection when the initial cracking occurs, and the height-to-length ratio of the wall. The maximum

permissible lateral deflection for design is the deflection at which the ultimate shear load is developed.

The ultimate shear resistance may be greater than, or equal to, the shear resistance at first cracking depending upon the steel reinforcement in the shear wall. It is recommended that sufficient steel be provided to make the ultimate resistance equal to the resistance developed at first cracking in the walls. If such a design proves inadequate the ultimate resistance may be increased by increasing the steel in the wall, or the resistance at first cracking may be increased by increasing the wall thickness.

Some elements of shear wall structures are subject to combined loading conditions. End walls and roof slabs are elements which are subject to blast pressures normal to their surfaces as well as loads in the plane of the slabs. Interior floor slabs of buildings which have openings in the walls are subject to the same type of combined loads. At certain locations on the slab the two effects tend to be additive, but, the exact stress condition is difficult to determine. It is recommended that where the effects are known to be additive that independent reinforcement be provided for each stress. An elastic design is recommended for deep beam action under lateral loads in the plane of the slabs. Elasto-plastic rather than plastic design should be used for roof slabs under blast loads acting normal to the roof surface thus providing a small factor of safety for the over-all behavior.

A similar condition exists in exterior shear walls that are subject to pressures acting normal to their surfaces in addition to shear loads transferred to them from the front and rear walls by the floor and roof slabs. For this case the maximum stresses for the two effects do not, in general, occur at the same location. Since the behavior of shear walls under combined loading is not well known, independent reinforcement should be provided for the two types of stress. In addition, a 50% reduction in the stiffness of end shear walls in the elastic range can be assumed because of the simultaneous normal and shear blast loading.

In determining stresses and deformations in shear wall structures it is frequently necessary to apply moment distribution procedures to elements

which are very deep in proportion to their length. It is also necessary in a frame analysis of shear walls with corridor openings to assume that a length of the panels near each joint of the frame has an infinite area and moment of inertia. Modified moment distribution formulas for these cases are presented in paragraph 9-04b to enable these problems to be handled through a variation of the familiar moment distribution procedure.

The inherent stiffness of shear wall structures makes them very sensitive to relatively small sliding and overturning actions of the structure as a unit. Therefore, it is necessary to determine the internal dynamic behavior of the shear wall elements simultaneously with an overturning and sliding analysis. The manner in which these analyses are performed is described in paragraph 9-06.

The determination of the shear wall resistance is explained in paragraph 9-05.

b. Modified Moment Distribution Formulas. In the derivation of the usual moment distribution formulas [5] shear deformations are neglected and only the moment deformation is considered. Modified formulas which do include the effect of shear deformations have been worked out [6]. A moment distribution procedure for members which have an infinite moment of inertia at the ends has frequently been used for design [7]. In the analysis of shear walls with large openings as frames it is necessary to make use of these modified formulas and procedures. Modified stiffness factors, carry-over factors, and sidesway moment formulas are presented on the following page for a prismatic member with an infinite moment of inertia and area at the ends of the member, and including the effect of shear deformations.

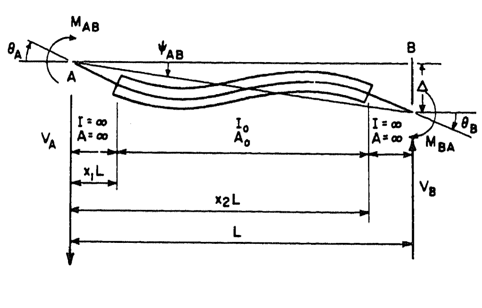


Figure 9.6. General prismatic member

A general prismatic member, AB , is illustrated in figure 9.6. The end rotations are θ_A and θ_B under the applied moments M_{AB} and M_{BA} and reactions V_A and V_B acting upon ends A and B , respectively. End B is displaced the distance Δ from its initial unloaded position, causing the chord rotation ψ_{AB}

The portion of the member lying between the sections defined by x_1L and x_2L has a constant moment of inertia and area of I_o and A_o, respectively. Between this portion of the member and the supports A and B, the member has an infinite moment of inertia and an infinite area.

The moment M_{AB} acting upon the end A of the member AB can be expressed in the following general form as a function of the joint rotations θ_A and θ_B, and of the chord rotation ψ_{AB}:

$$M_{AB} = \frac{EI_o}{L}\left[C_1\theta_A + C_2\theta_B - (C_1 + C_2)\,\psi_{AB}\right] \tag{9.1a}$$

where

$$C_1 = \left(\frac{a_3'}{a_2 - a_3'}\right) C_2 \quad \text{and} \quad a_1 = x_2 - x_1$$

$$a_2 = \frac{1}{2}\left(x_2^{\,2} - x_1^{\,2}\right)$$

$$C_2 = \frac{a_2 - a_3'}{a_1 a_3' - (a_2)^2} \quad \text{and} \quad a_3 = \frac{1}{3}\left(x_2^{\,3} - x_1^{\,3}\right)$$

$$a_3' = a_1 S + a_3$$

$$S = \frac{EI_o}{L^2 G\, A_o}$$

E = modulus of elasticity

$G = \dfrac{E}{2(1 + \nu)}$ = modulus of elasticity in shear

ν = Poisson's ratio (ν = 0.1 for concrete)

I_o = moment of inertia of member cross section

A_o = area in shear of member cross section

The moment M_{BA} acting upon the end B of member AB can be expressed in a similar form as a function of θ_A, θ_B, and ψ_{AB}

$$M_{BA} = \frac{EI_o}{L}\left[C_3\theta_B + C_2\theta_A - (C_3 + C_2)\,\psi_{AB}\right] \tag{9.1b}$$

where

$$C_3 = \left(\frac{a_1 - 2a_2 + a_3'}{a_2 - a_3'}\right) C_2$$

The stiffness factors at joints A and B of member AB are, respectively:

$$K_{AB} = \frac{C_1}{4}\left(\frac{I_o}{L}\right) \qquad K_{BA} = \frac{C_3}{4}\left(\frac{I_o}{L}\right) \tag{9.2a}$$

The stiffness factors as used in moment distribution solutions are proportionality factors. The stiffness factor K_{AB} for example is proportional to the moment M_{AB} required to produce a unit rotation of joint A of member AB ($\theta_A = 1$).

The carry-over factors from joint A to joint B and from joint B to joint A are, respectively:

$$C.O._{AB} = \frac{C_2}{C_1} = \frac{a_2 - a_3'}{a_3'} \qquad C.O._{BA} = \frac{C_2}{C_3} = \frac{a_2 - a_3'}{a_1 - 2a_2 + a_3'} \tag{9.2b}$$

The carry-over factors give the proportion between the moments at each end of a member due to a unit joint rotation at one end. For instance, $C.O._{AB}$ is the ratio M_{BA}/M_{AB} when M_{BA} and M_{AB} are the result of a joint rotation at A .

The fixed-end moments due to a chord rotation, ψ_{AB} , of member AB without joint rotations at A and B ($\theta_A = \theta_B = 0$) are given by the following equation:

$$\left.\begin{aligned} FEM_{AB} &= -(C_1 + C_2)\frac{EI_o}{L}\psi_{AB} = -4E\,(1 + C.O._{AB})\,K_{AB}\psi_{AB} \\ FEM_{BA} &= -(C_3 + C_2)\frac{EI_o}{L}\psi_{BA} = -4E\,(1 + C.O._{BA})\,K_{BA}\psi_{BA} \end{aligned}\right\} \tag{9.3}$$

where

$$\psi_{AB} = \psi_{BA}$$

9-05 <u>RESISTANCE OF SHEAR WALLS.</u> The resistance function of a shear wall is an expression relating its lateral deformation to the resistance which the wall develops to resist that deformation. For example, the idealized resistance function of a simple one-story shear wall is illustrated in figure 9.7.

The total wall deflection includes both moment and shear

deformations. Portions of the front and back walls act as flanges on the shear wall web and will affect the moment deformation, but only the shear wall web section contributes to the shear deformation. The modified moment distribution formulas given in paragraph 9-04b can be used to determine the stiffness factors. The procedure recommended in paragraph 9-04a is to proportion the steel in the wall so that $R_c = R_u$, and hence $k_2 = 0$. The remainder of this presentation of resistance functions will be limited to a discussion of the case where $R_u = R_c$. These results may be applied to other cases by substituting an equivalent average resistance function with $R_u = R_c$.

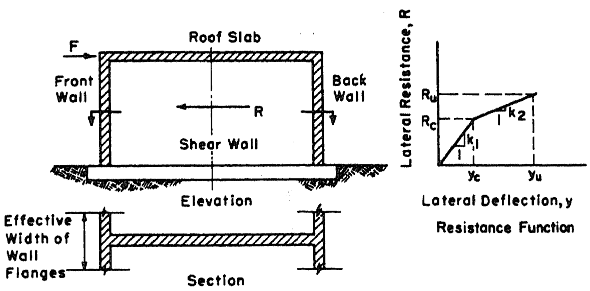


Figure 9.7. Idealized resistance function for a one-story shear wall

The resistance functions for a single two-story shear wall such as illustrated in figure 9.8 are as follows:

Case 1: $R_1 = k_{11}y_1 + k_{12}y_2 \qquad |R_1| \leq (R_1)_c$

$R_2 = k_{21}y_1 + k_{22}y_2 \qquad |R_2| \leq (R_2)_c$

Case 2: $R_1 = C_1 + k'_{11}y_1 \qquad |R_1| \leq (R_1)_c$

$R_2 = (R_2)_c \qquad |R_2| = (R_2)_c$

Case 3: $R_1 = (R_1)_c \qquad |R_1| = (R_1)_c$

$R_2 = C_2 + k'_{22}(x_2 - x_1) \qquad |R_2| \leq (R_2)_c$

k_{ij} (where i = 1, 2 and j = 1, 2) = wall resistance in i-th story developed by unit lateral deflection of top of j-th story wall (kips/ft)

k_{11} = wall resistance in first story developed by unit lateral deflection of top of first-story wall (kips/ft)

y_i = lateral deflection of top of i-th story wall (ft)

R_i = lateral resistance developed in i-th story wall (kips)

$(R_i)_c$ = lateral resistance of i-th story wall at which initial cracking occurs (kips)

As the maximum resistance $R_u = R_c$ is reached in any story, the

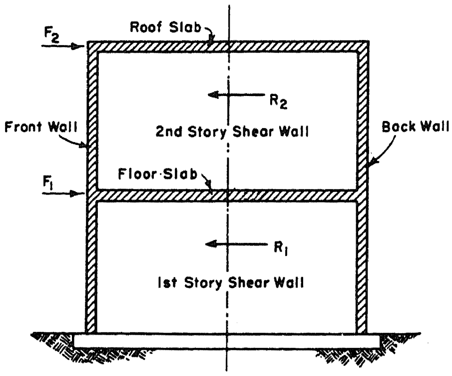


Figure 9.8. Two-story shear wall

resistance functions are altered to reflect the plastic stress condition (no further increase in moment and shear stresses) in that particular story wall under further deformation.

In general, the shear wall problem is not as simple as illustrated in figures 9.7 and 9.8. Many shear wall structures have longitudinal corridors which require openings in the shear walls (see fig. 9.9a). Such walls can be analyzed as a rigid frame, using the modified moment distribution method presented in paragraph 9-04b. The maximum lateral resistance of each story is determined by the strength of the individual shear wall components. The parts of the wall and foundation which join the two portions of the shear wall are made strong enough so that the initial cracking will occur in the shear wall panels each side of the openings, thus developing the full strength of the shear wall.

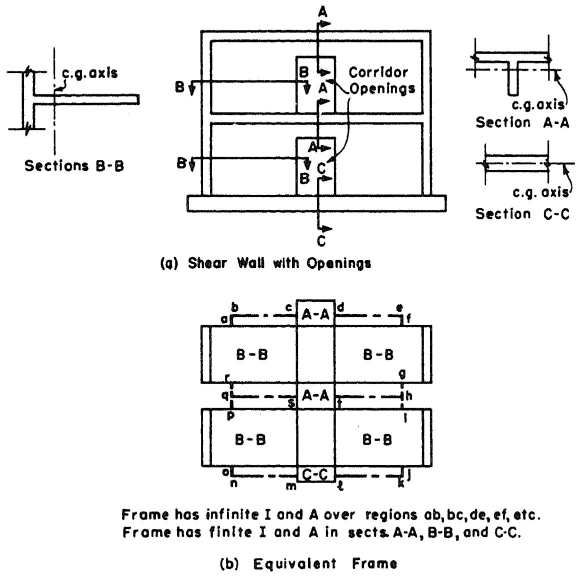


Figure 9.9. Shear wall with openings and equivalent frame

In applying the moment distribution method of paragraph 9-04b to the solution of a shear wall with openings, the equivalent frame axes should be chosen to coincide with the centroidal axes of the various structural sections around the openings in the wall as indicated in figure 9.9. The portion of each member of the frame which lies between the edge of the opening and the corner of the frame is assumed to have an infinite moment of inertia I and area A . In the usual building frame the members are relatively slender and it is therefore not necessary to consider the change in the moment of inertia near the frame joints. The shear wall elements, however, are very thick

relative to their length and the interference of the members at the frame joints must be handled as described above by assuming the overlapping portions of the members to have infinite stiffness. Thus the frame analysis includes only the shear and moment distortions of the portions of the wall adjacent to the openings. The horizontal shear distortion of the wall between the openings and between the upper opening and the roof of the structure may be superimposed upon the frame analysis if desired.

9-06 SLIDING AND OVERTURNING ANALYSES. a. General. The investigation of foundations for sliding and overturning is discussed in paragraph 6-31 of EM 1110-345-416. In the design of shear wall structures it is particularly important that this mode of behavior be investigated, not only to obtain the magnitude of the overturning and sliding but primarily to determine the effect upon the shear wall response. Shear walls are very stiff structural elements and their behavior is greatly affected by the inertial forces caused by small sliding and overturning tendencies. For this reason the design procedure in paragraph 9-07 requires consideration of the rigid body sliding and overturning in the preliminary design and a combined dynamic, sliding, and overturning analysis in the final design.

b. Rigid Body Analysis. In paragraph 6-31 sliding and overturning of a rigid body are discussed and equations are presented by which the magnitude of the sliding and overturning may be obtained. The same equations are presented in this paragraph and developed further in paragraph 9-06c to include the internal deformation of the structure as a phenomenon occurring simultaneously with the sliding and overturning.

For structures located on soil, the structure is considered to rotate about the centroidal axis through the base of the footings and sliding and overturning are analyzed simultaneously. Figure 9.10 illustrates the forces acting

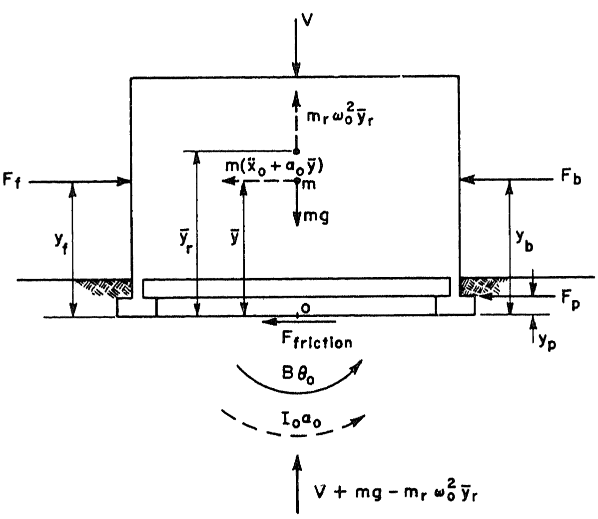


Figure 9.10. Forces acting upon structure as a rigid body

upon the structure as a rigid body. The structure rotates about the centroidal axis through the point "O" at the base of the footings. The angular acceleration of the structure about the axis "O" is given by:

$$\alpha_o = \frac{M_o - F_o\bar{y}}{I_o - m\bar{y}^2} \tag{9.4}$$

where

α_o = angular acceleration of structure about axis of rotation "O"

ω_o = angular velocity of structure about axis of rotation "O"

θ_o = angular displacement of structure about axis of rotation "O"

M_o = moment of all external forces about axis of rotation "O"

I_o = mass moment of inertia of structure about axis of rotation "O"

F_o = summation of all external horizontal forces applied to the structure including foundation reactions

m = total moving mass of structure and earth enclosed between footings

$\bar{y}$ = vertical distance from axis of rotation "O" to centroid of total moving mass, m

m_r = mass of structure considered to rotate as well as translate

$\bar{y}_r$ = vertical distance from axis of rotation "O" to centroid of rotating mass, m_r

The horizontal acceleration of the axis "O" is given by:

$$\ddot{x}_o = \frac{F_o}{m} - \alpha_o\bar{y} \tag{9.5}$$

where

$\ddot{x}_o$ = horizontal acceleration of axis of rotation "O"

v_o = horizontal velocity of axis of rotation "O"

x_o = horizontal displacement of axis of rotation "O"

The force components necessary for the computation of the forces F_o and the moment M_o are illustrated in figure 9.10 for a symmetrical structure. The computation of the sliding and rotation of the structure as a function of time is accomplished by concurrent numerical integrations of equations (9.4) and (9.5) using the methods outlined in paragraph 5-08. If the structure does not slide or stops sliding $(v_o = 0)$ at some point in

the analysis, equation (9.4) must be revised to the following form

$$\alpha_o = \frac{M_o}{I_o} \tag{9.6}$$

For the case of no sliding, only the solution by numerical integration of equation (9.6) is necessary. When the structure does not slide the velocity v_o of the axis of rotation "O" is equal to zero and the acceleration $\ddot{x}_o$ is equal to zero. Setting equation (9.5) equal to zero the following sliding criterion is obtained:

$$\left.\begin{array}{l}\text{Sliding occurs when } F_o > m\bar{y}\alpha_o \\ \text{When } v_o = 0 \text{ and } F_o \text{ is not greater than } m\bar{y}\alpha_o\text{, then } F_o = m\bar{y}\alpha_o\end{array}\right\} \tag{9.7}$$

The value thus obtained for F_o, the summation of all external horizontal forces applied to the structure including the foundation reactions, may be used to determine the magnitude of the horizontal foundation reactions developed when the structure does not slide. These reactions are necessary to compute the value of M_o for the solution of equation (9.6).

c. <u>Simultaneous Dynamic Sliding and Overturning Analysis.</u> The simultaneous dynamic sliding and overturning analysis involves the horizontal deformation of the shear walls in addition to rigid body modes of behavior. The forces acting upon a two-story structure are illustrated in figure 9.11. In a manner similar to that for the rigid body analysis, the structure is assumed to rotate about point "O," the central transverse axis at the base of the footings. The angular acceleration of the structure about "O" is given for a structure of n-stories by:

$$\alpha_o = \frac{M_o - (F_1 - R_1)y_1 - (F_2 - R_2)y_2 - (F_3 - R_3)y_3 \cdot \cdot \cdot \cdot - (F_n - R_n)y_n}{I_o - m_1y_1^2 - m_2y_2^2 - m_3y_3^2 \cdot \cdot \cdot \cdot - m_ny_n^2} \tag{9.8}$$

where

α_o = angular acceleration of structure about axis of rotation "O" of structure

M_o = moment of all external forces about axis of rotation "O"

I_o = mass moment of inertia of structure about axis of rotation "O"

F_i $(i = 1,2....n)$ = summation of all external horizontal forces applied to mass m_i , including foundation reactions

R_i $(i = 1,2....n)$ = horizontal resistance developed by structure at location of mass m_i due to internal structural deformation

The horizontal acceleration of point "O" is given by:

$$\ddot{x}_o = \frac{(F_1 - R_1)}{m_1} - \alpha_o y_1 \tag{9.9}$$

where

$\ddot{x}_o$ = horizontal acceleration of axis of rotation "O"

The relative horizontal accelerations of the shear walls are given by:

$$\ddot{x}_i = \frac{(F_i - R_i)}{m_i} - (\ddot{x}_o + \alpha_o y_i) \tag{9.10}$$

where

$\ddot{x}_i$ $(i = 2,3....n)$ = horizontal acceleration of the mass at floor level "i" with respect to the acceleration caused by the rigid body sliding and overturning about axis "O"

$\dot{x}_i$ $(i = 2,3....n)$ = horizontal velocity of the mass at floor level "i" with respect to the velocity caused by the rigid body sliding and overturning about axis "O"

x_i $(i = 2,3....n)$ = horizontal displacement of the mass at floor level "i" with respect to the displacement caused by the rigid body sliding and overturning about axis "O"

The force components used for the computation of the forces F_i and the moment M_o are shown for a symmetrical structure in figure 9.11. The computation of the shear wall displacements and the sliding and rotation is accomplished by a numerical integration of equations (9.8), (9.9), and (9.10). The deflections must all be computed simultaneously. If the structure does not slide or stops sliding ($\dot{x}_o = 0$) at some point in the analysis, equation (9.8) becomes:

$$\alpha_o = \frac{M_o - (F_2 - R_2)y_2 - (F_3 - R_3)y_3 - \dots - (F_n - R_n)y_n}{I_o - m_2 y_2^2 - m_3 y_3^2 - \dots - m_n y_n^2} \tag{9.11}$$

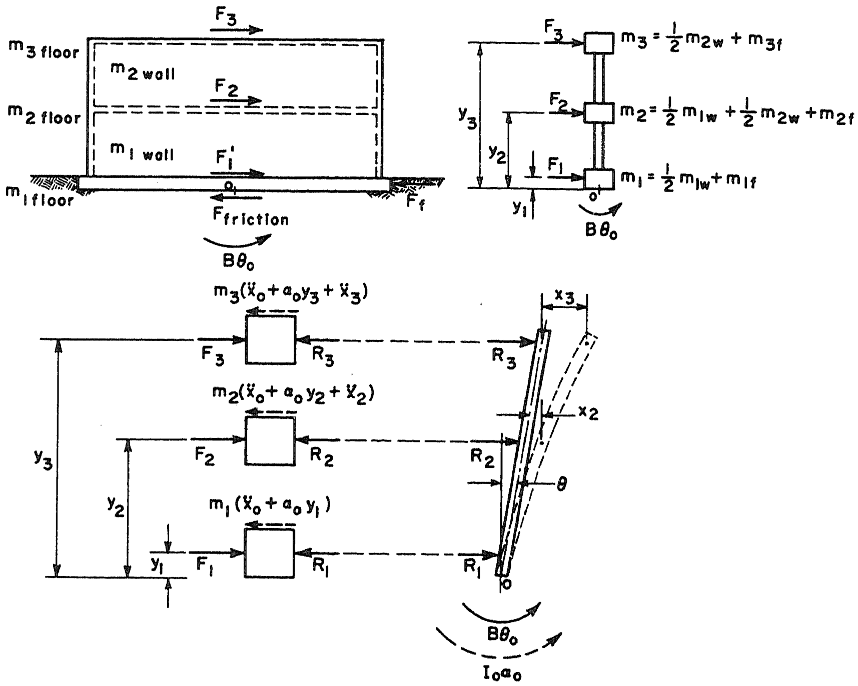


Figure 9.11. Forces acting upon two-story structure

For the case of no sliding, only the solution by numerical integration of equations (9.10) and (9.11) is necessary. When the structure does not slide the velocity v_o of the axis of rotation "O" is equal to zero and the acceleration $\ddot{x}_o$ is equal to zero. Setting equation (9.9) equal to zero, one obtains the following criteria:

$$\left.\begin{array}{l} \text{Sliding occurs when } (F_1 - R_1) > m_1 y_1 \alpha_o \\ \text{When } v_o = 0 \text{ and } (F_1 - R_1) \text{ is not greater than } m_1 y_1 \alpha_o, \\ \quad \text{then } (F_1 - R_1) = m_1 y_1 \alpha_o \end{array}\right\} \qquad (9.12)$$

Care must be taken in the computation of I_o , the mass moment of inertia of the structure about the axis of rotation "O." Because of the manner of construction of the structure it is often desirable to assume that some portions of the structure (such as the shear walls, front and back walls, and the edges of the roof supported by the walls) rotate as well as translate, whereas other portions (isolated footings and the columns and the roof supported by them) translate without rotating. The computation of I_o is thus the determination of the polar moment of inertia about "O" of those masses which rotate and translate, and the

moment of inertia about a horizontal plane through "0" of those masses which only translate.

9-07 GENERAL DESIGN PROCEDURE. The general design procedure for a shear wall structure built to withstand air blast loading is as follows:

Step 1. Determine the design criteria. These consist of the blast design loads, allowable stresses, clearances required, and the minimum allowable sizes of the structural elements. The minimum sizes may be based upon the required static load capacity, building code specification, or practical construction requirements. A preliminary layout of the structure at this point is very helpful.

Step 2. Design the roof and floor systems, columns, and column footings to carry the maximum vertical blast loads or conventional design loads as applicable. The critical blast loading on the roof is generally caused by Zone 1 roof loading with the blast wave traveling parallel to the long side of the structure. The roof dynamic reactions under the average Zone 3 roof loading (computed in Step 3) are determined for later use in the combined dynamic, sliding, and overturning shear wall analysis (Step 8). In designing the roof system one should keep in mind that the roof and floor slabs act as deep beams to transmit the horizontal front and back wall dynamic reactions to the top of the shear walls. The maximum stresses under the combined slab and deep beam actions do not occur together and no interaction formula is used in the design. Although separate steel reinforcement is provided for the two types of behavior, the designer has the option of designing the roof slab under the vertical roof overpressure for elastic and elasto-plastic action only. A safety factor is thus provided by the additional slab deflection possible in the plastic range in case the roof slab is overloaded by the simultaneous deep beam action.

Step 3. Determine the over-all dimensions of the structure and compute the average front wall, back wall, and Zone 3 roof loadings for the blast wave traveling normal to the long side of the structure.

Step 4. Design the exterior walls of the structure for the front wall loading and determine the lateral dynamic reactions of the front and back walls on the structure. Design the exterior wall footings.

Step 5. Perform a rigid body simultaneous sliding and overturning

analysis of the structure. Use the average front wall, back wall, and Zone 3 roof loadings computed in Step 3 for this analysis. Check the maximum soil pressures developed under the footings and revise footing sizes if necessary.

Step 6. Design the shear walls to carry the front wall dynamic reactions combined with the inertial forces in the structure due to the rigid body accelerations computed in Step 5. It is assumed that the shear walls all deflect the same distance, hence the total lateral load on each shear wall is proportional to the relative stiffness of the shear walls. This assumption, which makes it possible to analyze the dynamic behavior of all shear walls simultaneously, is equivalent to the standard assumption in building design that the roof and floor slabs act as rigid diaphragms. The stiffness of the end shear walls in the elastic range is arbitrarily reduced fifty percent because of the effect of the simultaneous normal blast loads acting upon the end walls.

Step 7. Perform a simultaneous dynamic shear wall, sliding, and overturning analysis to determine the final structural behavior. Check the maximum soil stresses and the maximum net lateral forces acting on the shear walls. Revise the footing and shear wall design if necessary.

Step 8. Design the roof and floor slabs as deep beams to carry the maximum net horizontal forces acting upon the shear walls as determined in Step 7. Design the front and back walls to carry the maximum net vertical roof and footing forces to the front and back edges, respectively, of the shear walls. The maximum shears and moments in the roof and wall elements as deep beams are determined by a moment distribution analysis. The horizontal roof deep beam reactions obtained from the moment distribution analysis are compared with the maximum shear wall reactions obtained in Step 7 to check the accuracy of the previous assumption. The shear walls are reinforced to resist the largest of the two stresses obtained in this step and in Step 7.

DESIGN EXAMPLE, ONE-STORY SHEAR WALL BUILDING

9-08 GENERAL. a. Statement of Problem. The design of a windowless one-story reinforced concrete shear wall building is presented to illustrate

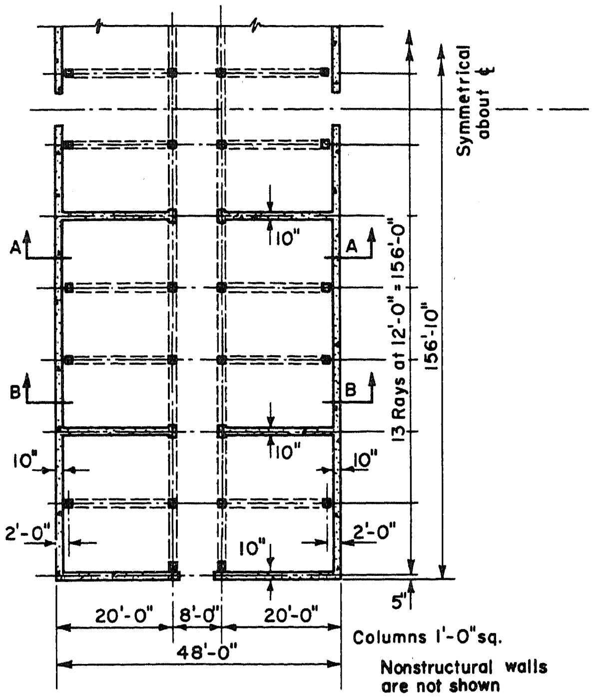


Figure 9.12. Plan of reinforced concrete shear wall building

the principles discussed in the preceding sections. The plan of the building is shown in figure 9.12. The foundation is shown in figure 9.13 and typical sections throughout the building are shown in figures 9.14 and 9.15. The roof is designed as a two-way slab over the office areas and as a one-way slab over the corridor. The roof is supported by girders and columns and by the shear walls and exterior walls as indicated. The location of the shear walls was chosen to suit the functional requirements of the building.

The air blast incident overpressure vs time curve which the building is designed to resist is plotted in figure 9.16. The air blast may approach the structure from any direction. The shock front is normal to the ground surface. The peak air blast overpressure is 10 psi and the duration of the positive phase is 0.71 sec (fig. 3.10).

The strength properties of the materials to be used are given on page 22. Intermediate grade reinforcing steel and a

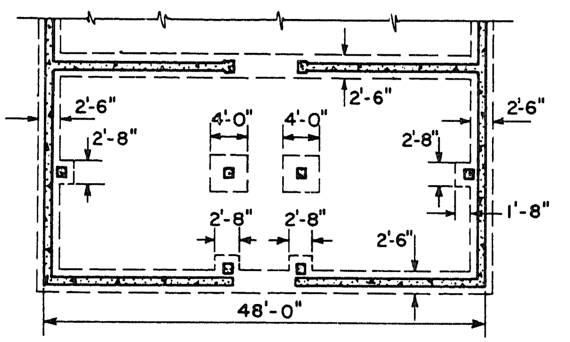


Figure 9.13. Typical foundation arrangement

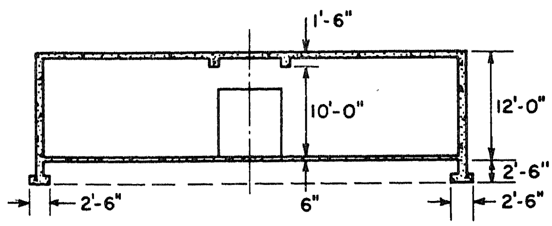


Figure 9.14. Section A-A of figure 9.12 at shear wall

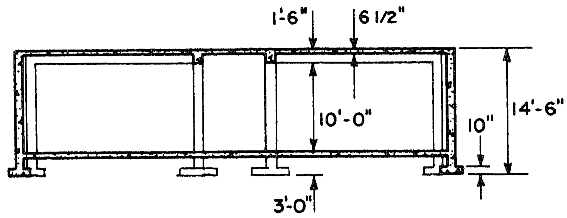


Figure 9.15. Typical section B-B of figure 9.12

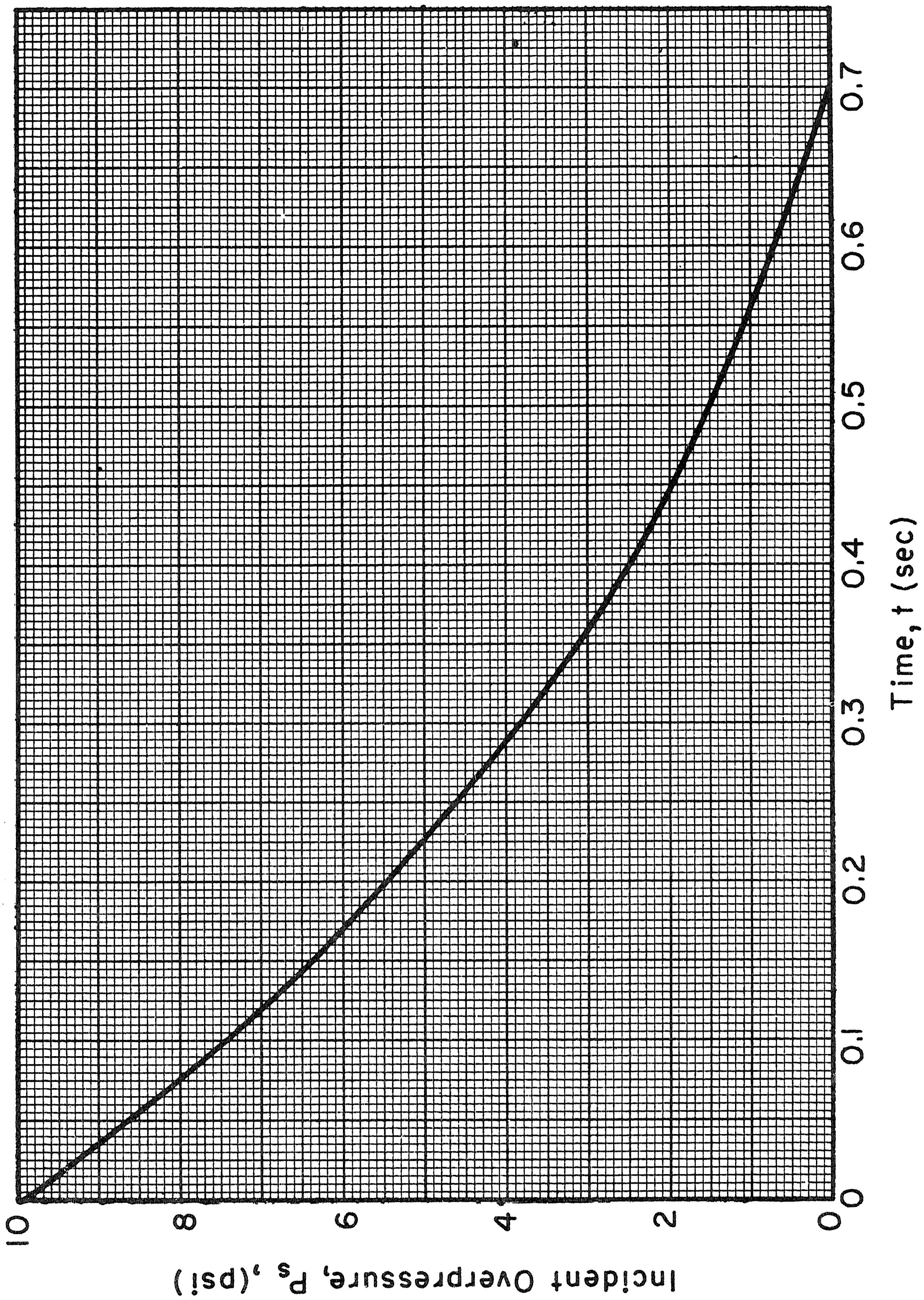


Figure 9.16. Air blast incident overpressure vs time curve

3000-psi concrete are to be specified. A thirty percent increase in strength of the materials is used because of the high rates of strain under dynamic loads. The notation used is that introduced in EM 1110-345-414. It will be noted that the strength values assumed for this example do not agree with the values recommended in EM 1110-345-414. The latter should be used for current projects.

$$f_y = 40{,}000 \text{ psi} \qquad f'_{dc} = 3{,}900 \text{ psi}$$
$$E_c = 3(10^6) \text{ psi} \qquad f'_c = 3{,}000 \text{ psi}$$
$$f_{dy} = 52{,}000 \text{ psi} \qquad n = 10$$

The structure is to be located upon a compact sand-gravel mixture having the following properties (see par. 4-15):

Normal load-bearing capacity = 10 kips/sq ft

Ultimate load-bearing capacity = 30 kips/sq ft

Modulus of elasticity = 40,000 psi

Coefficient of friction (soil on soil) = 0.75

Coefficient of friction (concrete on soil) = 0.75

Unit weight of soil = 100 lb/cu ft

Normal component of passive pressure coefficient, $K_{P\phi} = 10$

b. Design Procedure. The design of the structural elements is accomplished in the following order in accordance with the procedures presented in paragraph 9-07:

9-09 Design of Roof Slabs
9-10 Design of Roof Girders
9-11 Column and Column Footing Design
9-12 Wall Slab Design
9-13 Rigid Body Overturning and Sliding Analysis
9-14 Preliminary Investigation of Shear Walls
9-15 Dynamic Overturning and Sliding Investigation
9-16 Roof and Floor Slab Design (Deep Beam Action)
9-17 Wall Analysis (Deep Beam Action)
9-18 Final Design of Shear Walls
9-19 Design Summary

9-09 DESIGN OF ROOF SLABS. a. Vertical Blast Loads on Roof. The direction of travel of the air blast wave is assumed to be normal to either wall of the building to determine the critical load for all elements.

Paragraph 3-09 indicates the variations in roof loading with direction of the blast wave by the use of zones where:

Zone 1 indicates full incident blast wave overpressure,

Zone 2 indicates slightly reduced incident blast wave overpressure,

Zone 3 indicates greatly reduced incident blast wave overpressure.

With the direction of travel of the blast wave normal to the long dimension of the building, it is apparent that the roof loading in the end bays (Zones 1 and 2) is greater than in the center bays (Zone 3), and the end roof panels are critical. However, with the air blast wave traveling in the direction normal to the short dimension of the building, the maximum loading (Zone 1) exists over the entire roof. Therefore, the roof slabs must all be designed to carry the Zone 1 loading which is the full incident blast wave overpressure illustrated in figure 9.16.

For the portion of the roof designed as a series of two-way slabs, it is necessary to consider the total load on each slab. The load curve for a typical slab is obtained by using the average Zone 1 pressure on the slab computed as explained in paragraph 3-09e. This average overpressure-time curve is plotted in figure 9.17.

For the portion of the roof designed as a one-way slab, it is necessary to consider only the instantaneous pressure on a one-foot strip of slab, using the local Zone 1 overpressure-time curve (fig. 9.16).

It is important to note that the average blast loading over the entire roof surface which takes into account the reduction of overpressure in Zone 3 (fig. 9.18) is used in the foundation design and overturning and sliding analysis because it gives the true total load on the roof. To assume the average Zone 1 loads over the entire roof would not be realistic for the sliding analysis because the total load and therefore the friction would be too great.

b. Design of Two-way Roof Slabs. The roof slab design in this step is based on only the blast loads normal to the slab. The deep beam action of the roof slab acting as a horizontal load-carrying element is temporarily disregarded because the lateral loads cannot be obtained until after

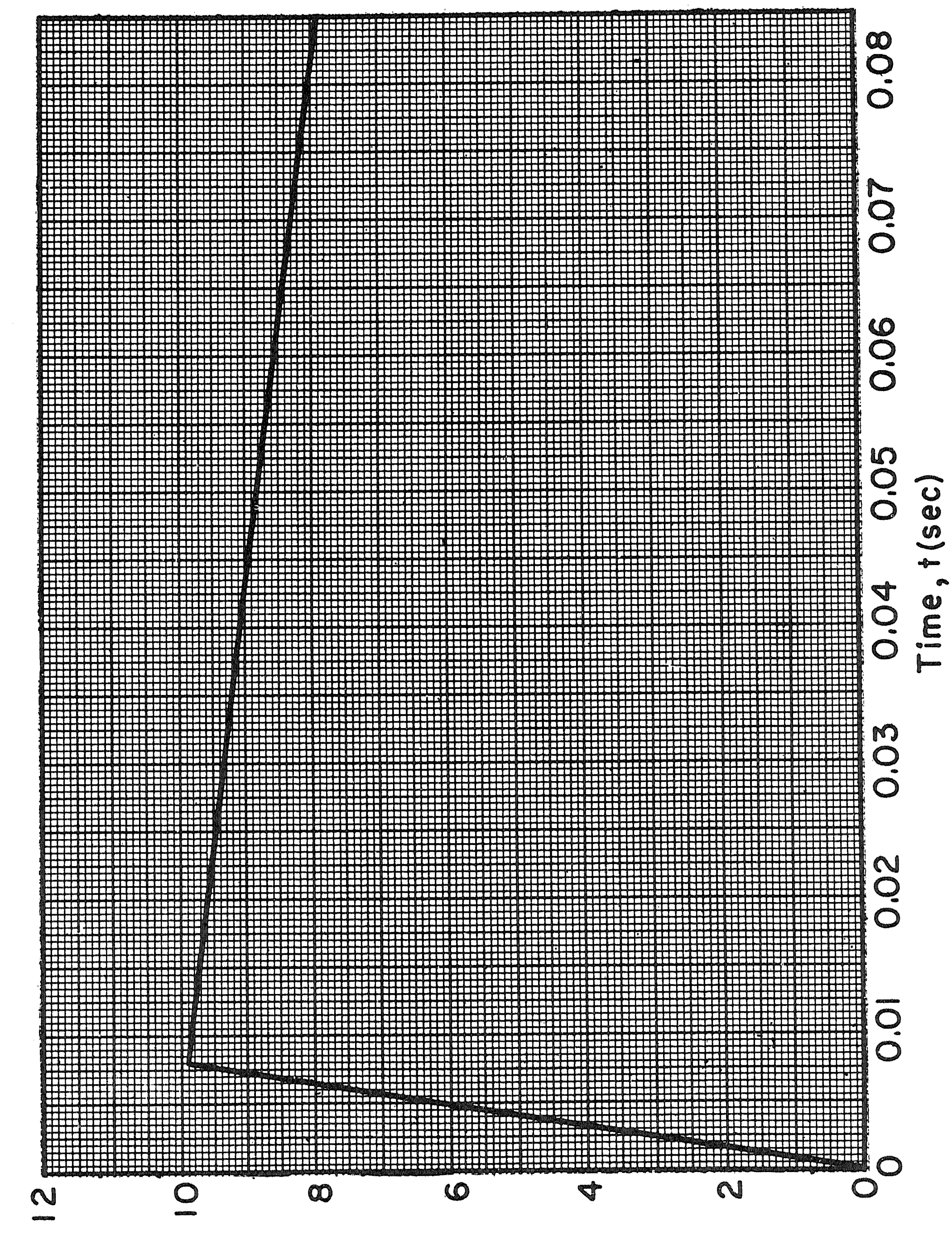


Figure 9.17. Average Zone 1 roof overpressure vs time curve on 11-ft-1-in.- by 18-ft-8-in. slab

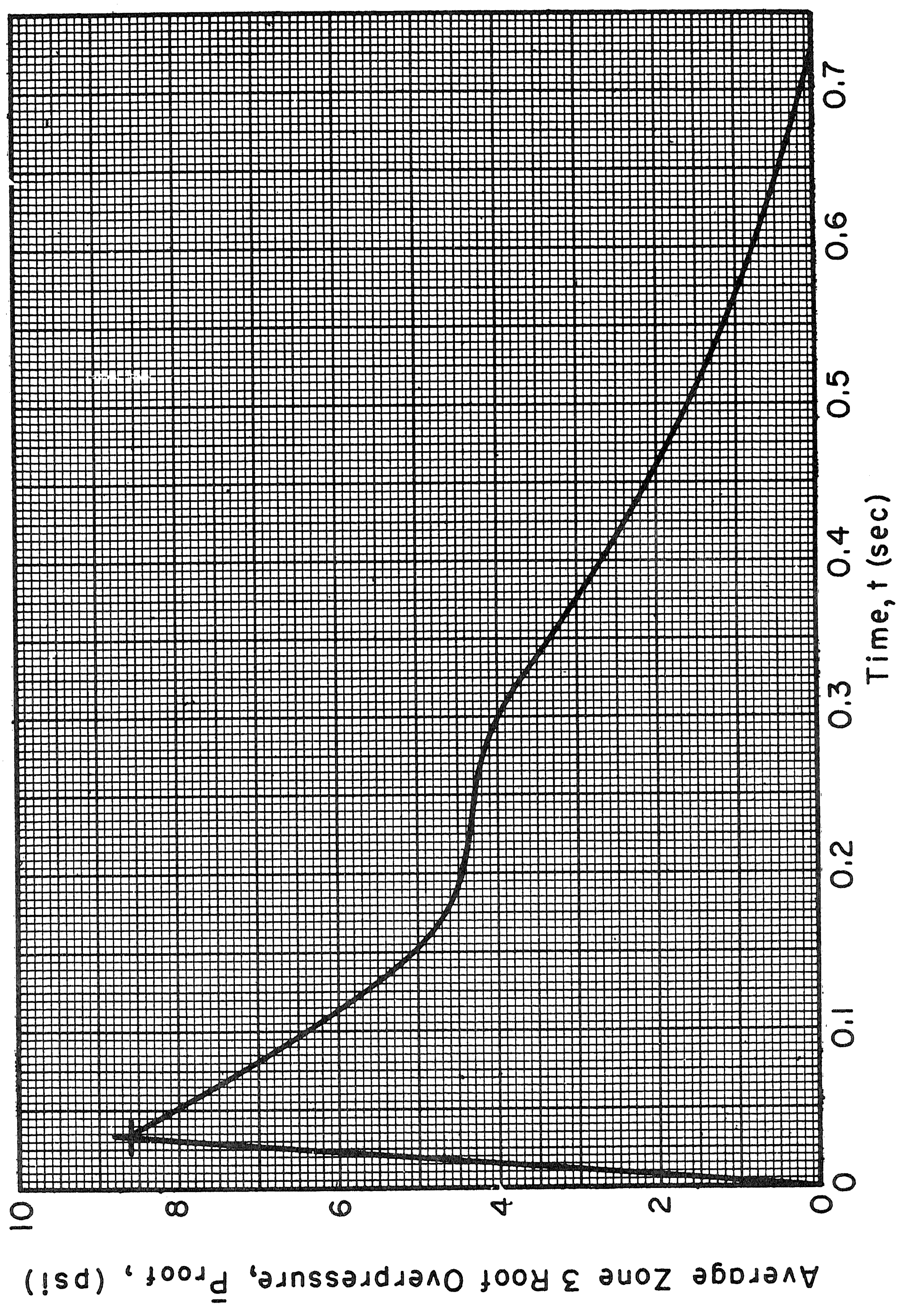


Figure 9.18. Average Zone 3 roof overpressure vs time curve

the front wall has been designed. (This will be considered later in paragraph 9-16.) The design is based on elastic and elasto-plastic action of a two-way slab so that additional capacity to absorb energy will be available in the purely plastic range. This will provide a safety factor against possible overloading of the slab due to the simultaneous deep beam action.

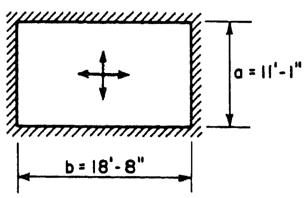
As stated in paragraph 9-09a, the vertical blast load is assumed to be uniformly distributed over the slab. The continuity of the slab over girders and transverse walls and at the wall-to-roof junctions justifies the assumption of full fixity at the four edges of the slab.

The first step in the procedure used in designing the roof slab is to assume a dynamic load factor (D.L.F.) for a preliminary determination of the required resistance. The slab dead weight is neglected in this calculation. From this resistance a first trial size is obtained. The properties of the actual slab are then computed from previously listed formulas as noted.

The equivalent single-degree dynamic system is obtained by applying coefficients taken from EM 1110-345-416 to the appropriate slab properties. The preliminary slab size is verified by a numerical integration procedure described in paragraph 5-08 using the Zone 1 load. In addition, the slab dynamic reaction, which is obtained from the numerical integration, is needed for the girder design which follows.

A final numerical analysis of the two-way roof slab is made to obtain the dynamic reactions from the average Zone 3 roof overpressure-time curve (fig. 9.18). These reactions are used later for the simultaneous dynamic, sliding and overturning analysis.

c. Design Conditions. Two-way slab, fixed four sides, elastic and elasto-plastic action, uniformly distributed load, equal strength at center and supports in both directions, Zone 1 loading condition.



d. Dynamic Design Factors (Table 6.2B).

$a/b = 11.08/18.67 = 0.593$, say 0.6

Elastic: $K_{LM} = 0.71$, $k_1 = 778\ EI_a/a^2$

$R_{1m} = 26.4\ M^o_{Psb}$

$V_A = 0.06P + 0.09R$, $V_B = 0.12P + 0.23R$

Elasto-plastic: $K_{LM} = 0.74$, k_{ep} 212 EI_a/a^2

$$R_m = \frac{1}{a}\left[12(M_{Pfa} + M_{Psa}) + 9.3(M_{Pfb} + M_{Psb})\right]$$

$$V_A = 0.04P + 0.11R,\ V_B = 0.09P + 0.26R$$

e. Required Slab Properties. Assume $p = 0.015$ in each direction in bottom of slab at center of span and top of slab at supports. Neglect compression steel in strength computation.

$$M^o_{Psb} = \frac{1}{11.08} M_{Pfb} = \frac{1}{11.08} M_{Psb} = \frac{1}{18.67} M_{Pfa} = \frac{1}{18.67} M_{Psa}$$

$$= p\, f_{dy}\, bd^2 \left(1 - \frac{p\, f_{dy}}{1.70\, f'_{dc}}\right) \qquad \text{(eq 4.17)}$$

$$= 0.015(52)\ \frac{12}{12}\ (d)^2 \left[1 - \frac{(0.015)52}{1.70(3.9)}\right] = 0.688\ d^2 \text{ kip-ft/ft}$$

$$R_{lm} = 26.4\ M^o_{Psb} = 26.4\ (0.688\ d^2) = 18.2\ d^2 \text{ kips (d in in.)}$$

$$R_m = \frac{1}{11.08}\left\{12\left[18.67(0.688\ d^2) + 18.67\ (0.688\ d^2)\right] + 9.3\left[11.08(0.688\ d^2) + 11.08(0.688\ d^2)\right]\right\}$$

$$= (27.8 + 12.8)\ d^2 = 40.6\ d^2 \text{ kips}$$

f. First Trial Section. Assume D.L.F. = 1.5 (approximate value for preliminary elasto-plastic design)

$$\text{Reqd } R_m = \frac{\text{(D.L.F.)(Max } \overline{P}_{roof} \text{ for Zone 1)(a)(b)}}{1{,}000}$$

$$= 1.5(9.88)144(11.08)\ 18.67/1{,}000 = 441 \text{ kips},\ 40.6\ d^2 = 441$$

Reqd $d = \sqrt{10.8} = 3.3$ in.

Reqd $A_s = pdb = 0.015(3.3)12 = 0.60$ in.^{2}/ft

Try #5 at 6 in., $A_s = 0.62$ in.^{2}/ft, $\Sigma o = 3.9$ in./ft

$t = 6$ in., negative $d = 3.5$ in., positive $d = 4$ in.

negative $p = (0.62)/3.5(12) = 0.0148$ at support

positive $p = (0.62)/4(12) = 0.0129$ at center

Use #5 at 12 in. for positive steel at support and for negative steel at center of span for reverse bending resistance.

$$M_{Pfa} = (p_{pos})f_{dy}(a)(d_{pos})^2 \left[1 - \frac{p_{pos}\ f_{dy}}{1.7\ f'_{dc}}\right]$$

$$= 0.0129\ (52)11.08(4)^2 \left[1 - \frac{0.0129(52)}{1.70(3.9)}\right] = 106.9 \text{ kip-ft}$$

$$M_{Pfb} = \left(\frac{b}{a}\right) M_{Pfa} = \frac{18.67}{11.08}\ (106.9) = 180.1 \text{ kip-ft}$$

$$M_{Psa} = (p_{neg})f_{dy}(a)(d_{neg})^2 \left[1 - \frac{p_{neg}\ f_{dy}}{1.7\ f'_{dc}}\right]$$

$$= 0.0148\ (52)11.08(3.5)^2 \left[1 - \frac{0.0148(52)}{1.70(3.9)}\right] = 92.4 \text{ kip-ft}$$

$$M_{Psb} = \left(\frac{b}{a}\right) M_{Psa} = \frac{18.67}{11.08}\ (92.4) = 155.9 \text{ kip-ft}$$

Weight of slab = 11.08(18.67)(150)/2(1,000) = 15.5 kips

$$R_{1m} = 26.4\ M^{o}_{Psb} - (\text{weight of slab})$$

$$= 26.4\ M^{o}_{Psb} - 15.5 = 26.4(155.9)/18.67 - 15.5 = 204 \text{ kips}$$

$$R_m = \frac{1}{a}\ 12(M_{Pfa} + M_{Psa}) + 9.3\ (M_{Pfb} + M_{Psb})$$

$$= \frac{1}{11.08} \left[12(106.9 + 92.4) + 9.3(180.1 + 155.9)\right] - 15.5 = 483 \text{ kips}$$

$$I_g = bt^3/12 = 12(6)^3/12 = 216 \text{ in.}^4/\text{ft}$$

Use positive steel at center of span for computation of I_t
$p = 0.0129$, $m = 10$, from table 1 in reference [8], $k = 0.395$.

$$I_t = b\ \frac{1}{3}(kd)^3 + np(1 - k)^2\ d^3$$

$$= (12) \left[(0.395)^3(4)^3/3 + 10(0.0129)(1 - 0.395)^2(4)^3\right]$$

$$= 12\ (1.317 + 3.025) = 52 \text{ in.}^4/\text{ft}$$

$$I_a = (I_g + I_t)/2 = (216 + 52)/2 = 134 \text{ in.}^4/\text{ft}$$

Elastic:

$k_1 = 778\ EI_a/a^2 = 778(3)10^3(134)/(11.08)^2\ 144 = 17{,}720$ kips/ft

$y_e = R_{1m}/k_1 = 204/17{,}720 = 0.0115$ ft

Mass of slab $= m_t = 15.5/32.2 = 0.481$ kip-sec^2/ft

Elastic $T_n = 2\pi\sqrt{K_{LM}\ m_t/k} = 6.28\sqrt{0.71(0.481)/17{,}720} = 0.028$ sec

Elasto-plastic:

$k_{ep} = 212\ EI_a/a^2 = \frac{212}{778}(17{,}720) = 4{,}840$ kips/ft

$y_{ep} = y_e + (R_m - R_{1m})/k_{ep} = 0.0115 + (483 - 204)/4{,}840 = 0.0691$ ft

g. Verification of Design and Reaction Determination by Numerical Integration. Use acceleration impulse extrapolation method (par. 5-08d).

$$y_{n+1} = 2y_n - y_{n-1} + \ddot{y}_n(\Delta t)^2 \qquad \text{(eq 5.49)}$$

$$\ddot{y}_n = (P_n - R_n)/K_{LM}m_t$$

Use $\Delta t = 0.0025$ sec (approximately 1/10 of elastic T_n)

Elastic Strain Range:

$(\Delta t)^2/K_{LM}m_t = (0.0025)^2/0.71(0.481) = 0.0000183$

$\ddot{y}_n(\Delta t)^2 = 0.0000183\ (P_n - R_n)$

$R_n = k_1 y_n = 17{,}720\ y_n$, $(y_n \leq 0.0115 = y_e)$

$P_n = 11.08(18.67)144\ \overline{P}_{roof}/1{,}000 = 29.8\ \overline{P}_{roof}$ kips

$\overline{P}_{roof}$ is obtained from figure 9.17

$V_A = 0.06P + 0.09R$, $V_B = 0.12P + 0.23R$

Elasto-plastic Strain Range:

$(\Delta t)^2/K_{LM}m_t = (0.0025)^2/0.74(0.481) = 0.0000176$

$\ddot{y}_n(\Delta t)^2 = 0.0000176\ (P_n - R_n)$

$R_n = R_{1m} + 4{,}840\ (y_n - y_e)$

$= 148 + 4{,}840\ y_n$, $(y_e = 0.0115 \leq y_n \leq 0.0691 = y_{ep})$

P_n is identical with elastic case

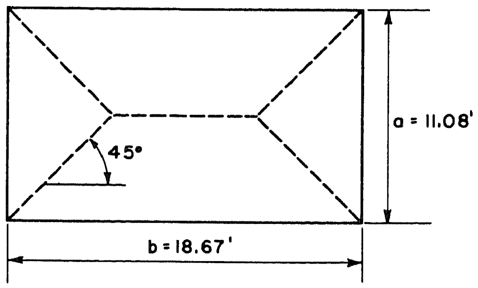
$V_A = 0.04P + 0.11R$, $V_B = 0.09P + 0.26R$

Table 9.1. Determination of Maximum Deflection and Dynamic Reactions for Two-way Roof Slab, First Trial (Zone 1 Loading)

t (sec)	$\bar{P}_{roof}$ (psi)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	Strain Range	V_A (kips)	V_B (kips)
0	0	0	0	91.7/6	0.000268	0		0	0
0.0025	3.08	91.7	4.75	86.9	0.00159	0.000268	e	5.93	12.09
0.005	6.16	184	37.80	146.2	0.00267	0.00213	e	14.45	30.80
0.0075	9.24	275	75.30	199.7	0.00366	0.00425	e	23.28	50.30
0.010	9.84	293	183.0	110.0	0.00201	0.01003	e	34.08	77.20
0.0125	9.75	290	234.2	55.8	0.00099	0.01782	e-p	37.40	87.10
0.015	9.68	288	277.0	11.0	0.00019	0.02660	e-p	32.00	97.90
0.0175	9.61	286	320.0	-34.0	-0.00060	0.03557	e-p	46.65	108.0
0.020	9.55	284	360	-76.0	-0.00139	0.04394*	e-p	51.00	119.2
0.0225	9.48	282	237**			0.03696	e	38.20	88.40

* Maximum deflection = 0.0439 ft < 0.0691 = x_p .
** $R_n = 360 - (0.04394 - y_n)\ 17{,}720 = 17{,}720\ y_n^p - 418$.

Maximum deflection is in the elasto-plastic range as desired, hence the section is satisfactory for bending.



h. Shear Strength and Bond Stress.

Area contributing dead load to "b" edges as outlined in diagram by dotted lines
$= 18.67(11.08) - (11.08)^2/2$
$= 145\ \text{ft}^2$

Dead load along one "b" edge
$= (\text{area}) \left(\frac{t}{12}\right) 150/2(1{,}000) = 145(75)/2(1{,}000) = 5.4$ kips

Area contributing dead load to "a" edges $= (11.08)^2/2 = 61\ \text{ft}^2$

Dead load along one "a" edge $\doteq 61(75)/2(1{,}000) = 2.3$ kips

Maximum total shear along edge "a" $= 2.3 + 51 = 53.3$ kips
$= 53.3/11.08$ kips/ft $= 4.82$ kips/ft

Maximum total shear along edge "b" $= 5.4 + 119 = 124.4$ kips
$= 124.4/18.67$ kips/ft $= 6.68$ kips/ft

Maximum shear intensity $= v = V/bjd = 6.68(1{,}000)/12 \left(\frac{7}{8}\right) 3.5$
$= 183$ psi

Allowable $v_y = 0.04\, f'_c + 5{,}000\, p + r\, f_y$ (eq 4.24a)

$= 0.04\ (3{,}000) + 5{,}000(0.0148) = 194$ psi

Maximum bond intensity $= u = V/\Sigma ojd = 6.68(1{,}000)/3.9 \left(\frac{7}{8}\right) 3.5$
$= 558$ psi

Allowable $u = 0.15\, f'_c$ (par. 4-09b)

$= 0.15\ (3{,}000) = 450 \text{ psi} < 558$ psi

Because of bond stress excess a thicker slab will be tried.

Use $t = 6\text{-}1/2$ in., #4 at 4 in., $\Sigma o = 4.7$ in., $A_s = 0.60 \text{ in.}^2/\text{ft}$

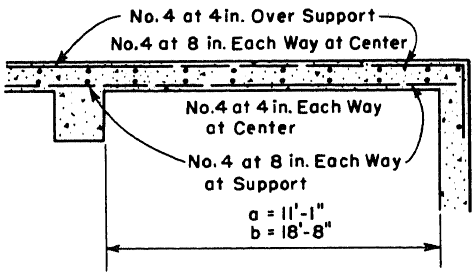
i. Revised Slab Properties.

$t = 6\text{-}1/2$ in., #4 at 4 in., $\Sigma o = 4.7$ in., $A_s = 0.60 \text{ in.}^2/\text{ft}$

negative $d = 4$ in., negative $p = (0.60)/4(12) = 0.0125$

positive $d = 4.5$ in., positive p
$= (0.60/4.5(12) = 0.0111$

Use #4 at 8 in. for compression steel at support and at center of span for reverse bending to provide resistance to rebound. Ignore this steel in strength and stiffness computations.



$$M_{Pfa} = (p_{pos})\, f_{dy}\, (a)(d_{pos})^2 \left[1 - \frac{p_{pos}\, f_{dy}}{1.7\, f'_{dc}}\right]$$

$$= 0.0111(52)\ 11.08(4.5)^2 \left[1 - \frac{0.0111(52)}{1.70(3.9)}\right] = 118.2 \text{ kip-ft}$$

$$M_{Pfb} = \left(\frac{b}{a}\right) M_{Pfa} = \frac{18.67}{11.08}\,(118.2) = 199.1 \text{ kip-ft}$$

$$M_{Psa} = (p_{neg})\, f_{dy}\, (a)(d_{neg})^2 \left[1 - \frac{p_{neg}\, f_{dy}}{1.7\, f'_{dc}}\right]$$

$$= 0.0125(52)\ 11.08(4)^2 \left[1 - \frac{0.0125(52)}{1.70(3.9)}\right] = 115.2 \text{ kip-ft}$$

$$M_{Psb} = \left(\frac{b}{a}\right) M_{Psa} = \frac{18.67}{11.08}\,(115.2) = 194.2 \text{ kip-ft}$$

Weight of slab $= 11.08(18.67)\ 150(6.5)/12(1{,}000) = 16.8$ kips

$$R_{1m} = 26.4\ M_{Psb} - \text{(weight of slab)}$$

$$= 26.4\ M^{o}_{Psb} - 16.8 = 26.4(194.2)/18.67 - 16.8 = 258 \text{ kips}$$

$$R_m = \frac{1}{a}\left[12(M_{Pfa} + M_{Psa}) + 9.3(M_{Pfb} + M_{Psb})\right] - \text{(weight of slab)}$$

$$= \frac{1}{11.08}\left[12(118.2 + 115.2) + 9.3(199.1 + 194.2)\right] - 16.8 = 566 \text{ kips}$$

$$I_g = bt^3/12 = 12(6.5)^3/12 = 274 \text{ in.}^4/\text{ft}$$

At center positive p = 0.0111, n = 10, from table 1 in reference [2]

$$k = 0.373$$

$$I_t = b\left[\frac{1}{3}(kd)^3 + np(1 - k)^2 d^3\right]$$

$$= 12\left[(0.373)^3(4.5)^3/3 + 10(0.0111)(1 - 0.373)^2(4.5)^3\right]$$

$$= 12\ (1.577 + 3.98) = 67 \text{ in.}^4/\text{ft}$$

$$I_a = (I_g + I_t)/2 = (274 + 67)/2 = 170 \text{ in.}^4/\text{ft}$$

Elastic:

$$k_1 = 778\ EI_a/a^2 = 778(3)10^3\ (170)/(11.08)^2\ 144 = 22{,}400 \text{ kips/ft}$$

$$y_e = R_{1m}/k_1 = 258/22{,}400 = 0.0115 \text{ ft}$$

$$\text{Elastic } T_n = 2\pi\sqrt{K_{LM} m_t/k} = 6.28\sqrt{0.71(0.521)/22{,}400} = 0.026 \text{ sec}$$

Elasto-plastic:

$$k_{ep} = 212\ EI_a/a^2 = \frac{212}{778}\ (22{,}400) = 6{,}100 \text{ kips/ft}$$

$$y_{ep} = y_e + (R_m - R_{1m})/k_{ep} = 0.0115 + (566 - 258)/6{,}100 = 0.0620 \text{ ft}$$

Mass of slab = m_t = (weight of slab)/g = 16.8/32.2
= 0.521 kip-sec^2/ft

j. <u>Verification of Design and Reaction Determination by Numerical Integration (Revised Slab).</u> Use acceleration impulse extrapolation method (par. 5-08d).

$$y_{n+1} = 2y_n - y_{n-1} + \ddot{y}_n(\Delta t)^2 \qquad \text{(eq 5.49)}$$

$$\ddot{y}_n = (P_n - R_n)/K_{LM}m_t$$

Δt = 0.0025 sec (approximately 1/10 elastic T_n)

Table 9.2. Determination of Maximum Deflection and Dynamic Reactions for Two-way Roof Slab, Second Trial (Zone 1 Loading)

t (sec)	P_{roof} (psi)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	Strain Range	V_A (kips)	V_B (kips)	V_B* (kips)
0	0	0	0	91.7/6	0.000258	0		0	0	0
0.0025	3.08	91.7	5.77	85.9	0.00145	0.000258	e	6.02	12.3	12.3
0.005	6.16	184	44.1	139.9	0.00236	0.00197	e	14.90	32.3	32.3
0.0075	9.24	275	83.4	191.6	0.00324	0.00372	e	24.0	52.1	52.1
0.010	9.84	293	195.0	98	0.00166	0.00871	e	35.1	80.2	92.5
0.0125	9.75	290	281.7	+8.3	0.00013	0.01536	e-p	42.6	99.4	131.7
0.015	9.68	288	323	-35	-0.00057	0.02214	e-p	47.1	110.1	162.2
0.0175	9.61	286	361	-76	-0.00123	0.02835	e-p	51.3	119.9	200.1
0.020	9.55	284	391	-107	-0.00173	0.03333	e-p	54.4	127.0	226.4
0.0225	9.48	282	411	-129	-0.00209	0.03658	e-p	56.5	132.4	242.4
0.025	9.41	280	418	-138	-0.00224	0.03774	e-p	57.2	134.2	254.1**
0.0275	9.35	278	393.8†	-115.8	-0.00196	0.03666	e	52.1	123.9	250.9
0.030	9.28	276	348	-72	-0.00122	0.03362	e	47.9	113.2	245.6
0.0325	9.20	274	230	+44	+0.000743	0.02936	e	37.0	85.8	220.0
0.035	9.15	272	153	+119	+0.00201	0.02584	e	30.0	67.8	191.7
0.0375	9.10	271	118	+153	+0.00258	0.02433	e	26.8	59.7	172.9
0.040	9.02	269	143	126	0.00213	0.02540	e	28.9	65.2	151.0
0.0425	8.97	268	214	54	0.00091	0.02860	e	35.3	81.3	149.1
0.045	8.90	265	305	-40	-0.00068	0.03271	e	43.3	101.9	161.6
0.0475	8.83	263	383	-120	-0.00203	0.03614	e	50.2	119.7	184.9
0.050	8.78	262	414	-152	-0.00257	0.03754	e	52.9	126.5	207.8
0.0525	8.70	260	388			0.03637	e	50.6	120.5	222.4

* $V_B' = V_B(t_n) + V_B(t_n - 0.0075)$. This is average load on girder at "b" end of slab span to be used later in girder design. Time for blast wave to traverse 11 ft 1 in. direction of slab = $2t_d$ = 11.08/1,400 = 0.008 ≈ 0.0075. (Velocity of blast wave = 1,400 fps.)

** Maximum V_B' = 254 kips. (See par. 9-10d.)

† $R_n = 418 - (0.03774 - y_n)(22,400) = -427 + 22,400\, y_n$

Elastic Strain Range:

$$(\Delta t)^2/K_{LM}m_t = (0.0025)^2/0.71(0.521) + 0.0000169$$

$$\ddot{y}_n(\Delta t)^2 = 0.0000169\ (P_n - R_n)$$

$$R_n = k_1 y_n = 22,400\ y_n$$

$$P_n = \frac{(a)(b)144}{1,000} \bar{P}_{roof} = 29.8 \bar{P}_{roof} \text{ kips}$$

$\bar{P}_{roof}$ is obtained from figure 9.17

$$V_A = 0.06P + 0.09R, \quad V_B = 0.12P + 0.09R$$

Elasto-plastic Strain Range:

$$(\Delta t)^2/K_{LM}m_t = (0.0025)^2/0.74(0.521) = 0.0000162$$

$$y_n(\Delta t)^2 = 0.0000162 \, (P_n - R_n)$$

$$R_n = R_{lm} + k_{ep} (y_n - y_e) = 258 + 6,100 \, (y_n - y_e)$$
$$= 188 + 6,100 \, y_n, \quad (y_e = 0.0115 \le y_n \le 0.0620 = y_{ep})$$

P_n is same as elastic case

$$V_A = 0.04P + 0.11R, \quad V_B = 0.09P + 0.26R$$

k. Shear Strength and Bond Stress of Revised Slab.

Area contributing dead load to "b" edges as shown in preceding sketch by dotted lines = 145 ft^2

Dead load along one "b" edge $= \left(\frac{\text{Area}}{2}\right)\left(\frac{t}{12}\right)\left(\frac{150}{1,000}\right) = \left(\frac{145}{2}\right)\left(\frac{6.5}{12}\right)\left(\frac{150}{1,000}\right)$ = 5.88 kips

Area contributing dead load to "a" edges = 61 ft^2

Dead load along one "a" edge $= \left(\frac{\text{Area}}{2}\right)\left(\frac{t}{12}\right)\left(\frac{150}{1,000}\right) = \left(\frac{61}{2}\right)\left(\frac{6.5}{12}\right)\left(\frac{150}{1,000}\right)$ = 2.47 kips

Maximum shear along edge "a" $= 2.47 + 57.2 = \frac{59.67}{11.08} = 5.40$ kips/ft

Maximum shear along edge "b" $= 5.88 + 134.2 = \frac{140.1}{18.67} = 7.50$ kips/ft

Maximum shear intensity $= v = \frac{V}{bjd} = \frac{7.50(1,000)}{12\left(\frac{7}{8}\right)(4.0)} = 178$ psi

Allowable $v_y = 0.04 \, f'_c + 5,000 \, p + r \, f_y$
$= 0.04(3,000) + 5,000(0.0125) = 182.5$ psi

182.5 > 178.0 psi OK for v

Maximum bond intensity $= u = \frac{V}{\Sigma o jd} = \frac{7.50(1,000)}{4.7(0.875)4.0} = 455$ psi

Allowable $u = 0.15 \, f'_c$ (par. 4-09b)
$= 0.15(3,000) = 450 \approx 455$ psi OK for u

l. Reaction Determination by Numerical Integration for Average Zone 3 Roof Loading. A dynamic analysis will be performed using average Zone 3

roof overpressure-time values (see fig. 9.18). The reactions obtained from this analysis are used in the simultaneous dynamic, sliding and overturning analysis.

Table 9.3. Determination of Dynamic Reactions for Two-way Roof Slab for the Sliding and Overturning Analysis (Zone 3 Loading)

t (sec)	$\bar{P}_{roof}$ (psi)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	Strain Range	V_A	V_B
0	0	0	0	17.9/6	0.0000504	0		0	0
0.0025	0.60	17.9	1.13	16.77	0.0002840	0.0000504	e	1.17	2.40
0.005	1.20	35.8	8.62	27.18	0.0004590	0.0003848	e	2.92	6.27
0.0075	1.85	55.2	26.40	28.80	0.0004870	0.0011782	e	5.69	12.68
0.010	2.50	74.5	55.00	19.50	0.0003300	0.0024586	e	9.42	21.58
0.0125	3.15	93.8	91.00	2.80	0.0000474	0.0040690	e	13.84	32.05
0.015	3.65	108.5	128.00	-19.50	-0.0003290	0.0057268	e	18.02	42.55
0.0175	4.30	128.0	158.00	-30.00	-0.0005060	0.0070556	e	21.68	51.80
0.020	5.00	149.0	176.00	-27.0	-0.0004560	0.0078784	e	24.75	58.40
0.0225	5.55	165.5	184.00	-18.5	-0.0003120	0.0082452	e	26.45	62.05
0.025	6.15	183.0	186.00	-3.0	-0.0000507	0.0083000	e	27.70	64.80
0.0275	6.80	202.0	188.20	+13.8	+0.0002340	0.0084041	e	28.90	67.60
0.030	7.45	222.0	195.50	+26.5	+0.0004480	0.0087422	e	30.90	71.60
0.0325	8.10	242.0	214.00	+28.0	+0.0004740	0.0095283	e	33.8	78.30
0.035	8.65	258.0	242.0	+16.0	+0.0002700	0.0107884	e	37.3	86.70
0.0375	8.55	255.0	263.2	-8.2	-0.0001330	0.0123185	e-p	39.2	91.30
0.040	8.45	252.0	271.6	-19.60	-0.0003180	0.0137156	e-p	39.00	93.20
0.0425	8.37	250.0	278.0	-28.00	-0.0004540	0.0147947	e-p	40.60	94.8
0.045	8.30	248.0	282.0	-34.00	-0.0005550	0.0154198	e-p	41.02	95.7
0.0475	8.15	243.0	282.5	-39.50	-0.0006400	0.0154899	e-p	40.92	95.4
0.050	8.09	241.0	269.5*			0.0149201	e	39.20	91.7

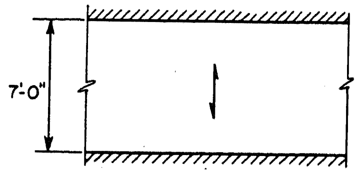
* $R_n = 282.5 - (0.01549 - y_n)22{,}400 = 22{,}400\, y_n - 64.5$.

m. Design of One-way Roof Slab. The design of the one-way roof slab over the corridors is similar to the two-way slab design (par. 9-09b) in the sense that the design is based on elastic and elasto-plastic action of a one-way slab so that additional capacity to absorb energy will be available in the purely plastic range to act as a safety factor against possible weakening of the slab due to the simultaneous deep beam action (investigated later in par. 9-16).

The load is assumed to be uniformly distributed over a 1-ft strip of slab. The continuity of the slab over the girders and adjacent two-way slabs justifies the assumption of full fixity at the two edges of the slab.

The properties of the slab will be obtained assuming the same thickness and one-direction steel identical with the adjacent two-way slabs. It is observable by the nature of the one-way action of slab and the orientation in the building that the critical blast load will be the instantaneous loading of a 1-ft strip with the local Zone 1 roof overpressure-time curve when the air blast is traveling in a direction normal to the end walls of the structure.

Under the average Zone 3 loading, the slab behavior is assumed to be static because of its extremely short period. Therefore no dynamic analysis is necessary.



n. Design Conditions. One-way slab, fixed at supports, elastic and elasto-plastic action, uniformly distributed load, equal strength at center and supports, Zone 1 loading condition. (See figure 9.16 for load curve.)

o. Dynamic Design Factors (Table 6.1b).

Elastic: $K_{LM} = 0.77$, $k_1 = 384\ EI/L^3$

$R_{1m} = 12M_{Ps}/L$

$V = 0.36R + 0.14P$

Elasto-plastic: $K_{LM} = 0.78$, $k_{ep} = 384\ EI/5L^3$

$R_m = \frac{8}{L}(M_{Ps} + M_{Pm})$

$V = 0.39R + 0.11P$

p. Slab Properties (Use same steel as for two-way slabs).

Use t = 6-1/2 in., #4 at 4 in., Σo = 4.7 in., A_s = 0.60 in.^{2}/ft

d = 4.5 in., p = 0.0111

Use same positive and negative steel throughout span to provide reverse bending strength for shear wall action. Use only tension steel in computation of strength and stiffness of slab.

$$M_{Ps} = M_{Pm} = p\ f_{dy}\left(\frac{b}{12}\right) d^2 \left[1 - \frac{p\ f_{dy}}{1.70\ f'_{dc}}\right]$$

$$= 0.0111(52)\frac{12}{12}(4.5)^2 \left[1 - \frac{0.0111(52)}{1.70(3.9)}\right] = 10.7 \text{ kip-ft/ft}$$

$$\text{Weight of slab} = \frac{1(7)150(6.5)}{12(1,000)} = 0.57 \text{ kips/ft}$$

$$R_{lm} = 12M_{Ps}/L - (\text{weight of slab}) = \frac{12(10.7)}{7} - 0.6 = 17.7 \text{ kips/ft width}$$

$$R_m = \frac{8}{L}(M_{Ps} + M_{Pm}) - (\text{weight of slab})$$
$$= \frac{8}{7}(10.7 + 10.7) - 0.6 = 23.8 \text{ kips/ft width}$$

$$I_g = bt^3/12 = \frac{12(6.5)^3}{12} = 274 \text{ in.}^4/\text{ft}$$

At center positive p = 0.0111, n = 10, from table 1 [8], k = 0.373

$$I_t = b\,\frac{1}{3}(kd)^3 + np\,(1 - k)^2d^3 = 12\,\frac{0.373^3(4.5^3)}{3}$$
$$+ 10(0.0111)(1 - 0.373)^2(4.5^3) = 12\,(1.577 + 3.98) = 67 \text{ in.}^4/\text{ft}$$

$$I_a = (I_g + I_t)/2 = \frac{274 + 67}{2} = 170 \text{ in.}^4/\text{ft}$$

Elastic:

$k_1 = 384\,EI_a/L^3 = 384(3)10^3(170)/(7)^3\,144 = 3,960$ kips/ft

$y_e = R_{lm}/k_1 = 17.7/3,960 = 0.0046$ ft

Mass of slab = m = (weight of slab)/g = 0.57/32.2 = 0.0177 kip-sec^2/ft

Elastic $T_n = 2\pi\sqrt{K_{LM}m/k_1} = 6.28\sqrt{\dfrac{0.77(0.0177)}{3,960}} = 0.011$ sec

Elasto-plastic:

$k_{ep} = k_1/5 = 3,960/5 = 792$ kips/ft

$y_{ep} = y_e + (R_m - R_{lm})/k_{ep} = 0.00446 + (24.4-17.7)/792 = 0.01291$ ft

q. <u>Verification of Design and Reaction Determination by Numerical Integration.</u> Use acceleration impulse extrapolation method (par. 5-08d).

$$y_{n+1} = 2y_n - y_{n+1} + \ddot{y}_n(\Delta t)^2 \qquad \text{(eq 5.49)}$$

$$\ddot{y}_n = (P_n - R_n)/K_{LM}m_t$$

$\Delta t = 0.00125$ sec (approximately 1/10 of elastic T_n)

Table 9.4. Determination of Maximum Deflection and Dynamic Reactions for One-way Roof Slab over Corridor (Zone 1 Loading)

t (sec)	P_s (psi)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	Strain Range	V (kips)	V_{ave}* (kips)
0	10.0	10.10	0	10.10	0.00136	0	e	1.41	0
0.00125	9.97	10.08	2.70	7.38	0.00092	0.00068	e	1.98	0.28
0.0025	9.95	10.05	9.02	1.03	0.00013	0.00288	e	4.65	0.84
0.00375	9.92	10.02	15.89	-5.87	-0.00073	0.00401	e	7.12	1.81
0.0050	9.89	10.00	18.12	-8.12	-0.00092	0.00501	e-p	8.17	3.11
0.00625	9.85	9.95	18.19	-8.24	-0.00093	0.00509**	e-p	8.20†	4.45
0.0075	9.80	9.90	14.29††	-4.39	-0.00055	0.00414	e	6.54	5.68
0.00875	9.78	9.87	8.34	+1.53	+0.00019	0.00264	e	4.38	6.32
0.0100	9.75	9.85	3.16	+6.69	+0.00083	0.00133	e	2.51	6.33
0.01125	9.71	9.80	1.25	+8.55	+0.00106	0.00085	e	1.82	5.70
0.0125	9.68	9.78	3.55	+6.23	+0.00077	0.00143	e	3.65	4.84
0.01375	9.65	9.75	8.89	+0.86	+0.00011	0.00278	e	4.56	4.22
0.0150	9.62	9.72	14.69	-4.97	-0.000615	0.00424	e	6.64	3.91
0.01625	9.58	9.68	17.99	-8.31	-0.00103	0.00508	e	7.82	4.22
0.0175	9.54	9.64	17.59	-7.95	-0.00099	0.00489	e	7.69	4.94
0.01875	9.50	9.60	12.59	-2.99	-0.00037	0.00371	e	5.88	5.70
0.0200	9.47	9.57	6.44	+3.13	+0.00038	0.00216	e	3.66	6.04
0.02125	9.43	9.53	1.81	+7.72	+0.00096	0.00099	e	1.98	5.49
0.0225	9.40	9.50	0.97	+8.53	+0.00106	0.00078	e	1.68	5.04
0.02375	9.37	9.47	4.34	+5.13	+0.00063	0.00163	e	2.88	4.22
0.0250	9.34	9.44	10.19	-0.75	-0.00009	0.00311	e	4.98	2.75
0.02625	9.30	9.40	15.69	-6.26	-0.00078	0.00450	e	6.96	3.60
0.0275	9.26	9.36	18.09	-8.73	-0.00108	0.00511	e	7.82	4.04
0.02875	9.23	9.33	16.29	-6.96	-0.00086	0.00464	e	7.17	4.99
0.0300	9.20	9.30	10.99	-1.66	-0.00021	0.00331	e	5.25	5.54
0.03125	9.16	9.26	4.89	+4.37	+0.00054	0.00177	e	3.05	5.88
0.0325	9.12	9.22	0.94	+8.28	+0.00102	0.00077	e	1.63	5.60
0.03375	9.09	9.19	1.81	+7.38	+0.00092	0.00099	e	1.93	4.90
0.0350	9.05	9.15	6.34	+2.81	+0.00034	0.00213	e	2.56	4.04
0.03625	9.01	9.11	12.19	-3.00	-0.00038	0.00301	e	6.16	3.56
0.0375	8.90	9.00	2.26	+6.74	+0.00083	0.00110	e	2.07	3.28
0.0400	8.80	8.90	-2.11	+6.79	+0.00084	0	e	0.48	2.80

* V_{ave} = weighted numerical average of V_n, V_{n-1}, V_{n-2}, V_{n-3}, V_{n-4}, V_{n-5}, and V_{n-6} = average blast load reaction per foot on one panel of longitudinal girder from corridor slab. This is used in girder design in paragraph 9-10h.

** Maximum deflection = 0.0051 < y_{ep} = 0.0129 ft, therefore design is satisfactory in bending.

† Maximum dynamic reaction = 8.20 kips.

†† $R_n = 18.19 - (0.00509 - y_n)3{,}960 = 3{,}960y_n - 2.11$.

Elastic Strain Range:

$(\Delta t)^2/K_{LM}m_t = 0.00125^2/0.71(0.0177) = 0.000124$

$\ddot{y}_n(\Delta t)^2 = 0.000124\ (P_n - R_n)$

$R_n = k_1 y_n = 3{,}960\ y_n,\ (y_n \leq 0.00446 = y_e)$

$P_n = \frac{wL(144)}{1{,}000} P_s = \frac{1.0(7.0)144}{1{,}000} P_s = 1.01\ P_s$

P_s is obtained from figure 9.16.

$V = 0.36R + 0.14P$

Elasto-plastic Strain Range:

$(\Delta t)^2/K_{LM}m_t = 0.00125^2/0.78(0.0177) = 0.0001132$

$\ddot{y}_n(\Delta t)^2 = 0.0001132\ (P_n - R_n)$

$R_n = R_{lm} + k_{ep}\ (y_n - y_e) = 17.70 + 790\ (y_n - y_e)$
$= 14.16 + 792\ y_n,\ (y_e = 0.00446 \leq y_n \leq 0.0121 = y_{ep})$

P_n is identical with elastic case

$V = 0.39R + 0.11P$

r. Shear Strength and Bond Stress.

Maximum shear = V = maximum dynamic reaction + dead load reaction
$= 8.20 + \frac{6.5}{12}\left(\frac{150}{1{,}000}\right)\frac{7}{2} = 8.47$ kips/ft

Shear intensity = $v = V/bjd = \frac{8{,}470}{12.0(0.875)4.5} = 179$ psi

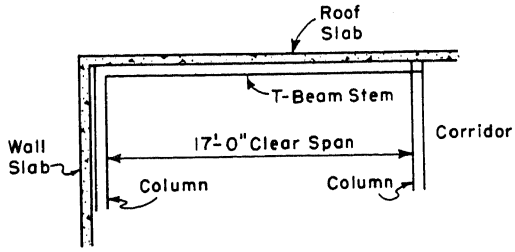
Allowable $v = 0.04\ f_c' + 5{,}000P + r\ f_y$, (eq 4.24a)
$= 0.04(3{,}000) + 5{,}000(0.0111) = 176.0$ psi

$179 \approx 176$ psi. Say OK

Maximum $u = V/\Sigma o jd = \frac{8{,}470}{4.7(0.875)4.5} = 458$ psi

Allowable $u = 0.15(3{,}000) = 450$ psi ≈ 458 psi. Say OK

9-10 DESIGN OF ROOF GIRDERS. a. Design Condition for Transverse Roof Girders. Simple span T-beam, elastic and plastic action, uniformly distributed mass and roof slab reaction under Zone 1 loading



condition, clear span = 17 ft 0 in., $x_m/x_e = \alpha\beta = 6$. (eq 6.91)

b. Design Loading. (See table 9.2.)

c. Dynamic Design Factors (Table 6.1A).

Elastic Strain Range:

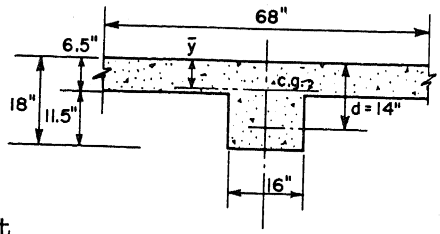
$K_{LM} = 0.78$, $R_m = 8M_p/L$, $k_1 = 384EI/5L^3$

$V = 0.39R + 0.11P$

Plastic Strain Range:

$K_{LM} = 0.66$, $V = 0.38R_m + 0.12P$

d. Girder Properties.



$b = b' + 8t$ (par. 4-10b)

$= 16 + 8(6.5) = 68$ in. $= 5.67$ ft

Assume D.L.F. = 1.0 (a reasonable assumption for plastic design with a slow rise time).

Maximum dynamic slab reaction acting on girder

$$= \frac{\text{clear span}}{\text{slab span}} (\text{maximum } V_B' \text{ from table 9.2}) = \frac{17}{18.67}(254) = 232 \text{ kips}$$

$$\text{Required maximum moment resistance} = \frac{LR}{8} = \frac{17(232)}{8} = 493 \text{ kip-ft}$$

$$M_P = A_s f_{dy}\left(d - \frac{a}{2}\right), \quad a = \frac{A_s f_{dy}}{0.85 f'_{dc} b}, \quad a < t \qquad \text{(eq 4.20)}$$

Assume $a = 6.5$ in., $d = 14$ in.

$$\text{Required } A_s = \frac{M_P}{f_{dy}\left(d - \frac{a}{2}\right)} = \frac{493(12)}{52\left(14 - \frac{6.5}{2}\right)} = 10.6 \text{ in.}^2$$

Try 8 #11 bars, $A_s = 12.48$ in.^{2}, $\Sigma o = 40.0$ in.

$$\text{Check: } a = \frac{A_s f_{dy}}{0.85 f'_{dc} b} = \frac{12.48(52)}{0.85(3.9)68} = 2.88 \text{ in.} < 6.5 \text{ in.} = t$$

$$M_P = A_s f_{dy}\left(d - \frac{a}{2}\right) = \frac{12.48(52)\left(14 - \frac{2.88}{2}\right)}{12} = 679 \text{ kip-ft}$$

$$\text{Weight of roof slab and beam} = \frac{150(17)\left[10.83(0.54) + 1.33(1.5)\right]}{1{,}000}$$

$= 20.0$ kips

$$R_m = \left(8M_P/L\right) - (\text{weight of roof slab and beam}) = \frac{8(679)}{17} - 20.0$$

$= 300$ kips

Gross I_g:

Area = $52(6.5) + 16(18) = 338 + 288$

$= 626 \text{ in.}^2$

$\bar{y} = \dfrac{338(3.25) + 288(9)}{626} = \dfrac{3,700}{626}$

$= 5.90$ in.

$I_g = 338\left(\dfrac{6.5^2}{12} + 2.65^2\right) + 288\left(\dfrac{18^2}{12} + 3.1^2\right) = 3,560 + 10,530$

$= 14,090 \text{ in.}^4$

Transformed I_t:

$n = 10,\ p = \dfrac{12.48}{68(14)} = 0.0131,\ np = 0.131,\ t/d = 6.5/14 = 0.465$

From Table 1 [8], $k = 0.397 < 0.465$

$I_t = \dfrac{b(kd)^3}{3} + n\, A_s\, (1 - k)^2\, d^2$

$= \dfrac{68(14^3)0.397^3}{3} + 10(12.48)(1 - 0.397)^2(14)^2$

$= 3,880 + 8,870 = 12,750 \text{ in.}^4$

$f_a = \frac{1}{2}\,(I_g + I_t) = \frac{1}{2}\,(14,090 + 12,750) = 13,420 \text{ in.}^4$

Elastic $k_1 = 384EI/5L^3 = \dfrac{384(3)10^3(13,420)}{5(17^3)144} = 4,360$ kips/ft

$y_e = R_m/k = 300/4,360 = 0.0688$ ft

$y_m = \alpha\beta\ y_e = 6y_e = 6(0.0688) = 0.413$ ft

Mass of roof slab and beam = m = $20.0/32.2 = 0.621 \text{ kip-sec}^2/\text{ft}$

$T_n = 2\pi\sqrt{K_{LM}m/k_1} = 6.28\sqrt{\dfrac{0.78(0.621)}{4,360}} = 0.066$ sec

e. <u>Verification of Design and Reaction Determination by Numerical Integration.</u> Use acceleration impulse extrapolation method (par. 5-08d).

$$y_{n+1} = 2y_n - y_{n-1} + \ddot{y}_n(\Delta t)^2 \qquad \text{(eq 5.49)}$$

$$\ddot{y}_n = (P_n - R_n)/K_{LM}m$$

Use Δt = 0.0025 sec (same as for roof slab)

Elastic Strain Range:

$(\Delta t)^2/K_{LM}m = 0.0025^2/0.78(0.621) = 0.0000129$

$\ddot{y}_n(\Delta t)^2 = 0.0000129\ (P_n - R_n)$ ft

$R_n = k_1 y = 4{,}360\ y_n$ kips, $(y_n \leq 0.0688 = y_e)$

$$P_n = V_B' + \frac{1(17)144}{1{,}000} P_s = V_B' + 2.44\ P_s$$

P_s is air blast incident overpressure vs time loading (fig. 9.16) applied with t_d = 0.0075 sec

V_B' is obtained from table 9.2

$V = 0.39R + 0.11P$

Plastic Strain Range:

$(\Delta t)^2/K_{LM}m = 0.0025^2/0.66(0.621) = 0.0000152$

$\ddot{y}_n(\Delta t)^2 = 0.0000152\ (P_n - R_n)$ ft

R_m = 300 kips

P_n is same as for elastic case above

$V = 0.38R_m + 0.12P$

$y_m = 6y_e = 0.413$ ft

Table 9.5. Determination of Maximum Deflection and Dynamic Reactions for Transverse Roof Girders (Zone 1 Loading)

t (sec)	P_s (psi)	$2.44P_s$ (kips)	V_B' (kips)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y (ft)	V (kips)
0	0	0	0	0	0	$\frac{12.2}{6}$	0.00003	0	0
0.0025	0	0	12.3	12.3	0.1	12.2	0.00016	0.000003	1.39
0.005	0	0	32.3	32.3	1.0	31.3	0.00040	0.00022	3.94
0.0075	10	24.4	52.1	76.5	3.5	73.0	0.00094	0.00081	9.76
0.010	9.9	24.2	92.5	116.7	10.2	106.5	0.00137	0.00234	16.78
0.0125	9.8	23.9	131.7	155.6	22.8	132.8	0.00171	0.00524	26.00
0.015	9.8	23.9	162.2	186.1	43.0	143.1	0.00185	0.00985	37.20
0.0175	9.7	23.6	200.1	223.6	71.0	152.6	0.00197	0.01631	52.20
0.020	9.6	23.4	226.4	249.8	108.0	141.8	0.00183	0.02474	69.4
0.0225	9.5	23.2	242.4	265.6	152.5	113.1	0.00146	0.03500	88.5
0.025	9.5	23.2	254.1	277.3	204.0	73.3	0.00095	0.04672	110.0
0.0275	9.4	22.9	250.9	273.8	259.0	14.8	0.00019	0.05939	131.0

Table 9.5 (Continued)

t (sec)	P_s (psi)	$2.44P_s$ (kips)	V_B' (kips)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y (ft)	V (kips)
0.030	9.3	22.7	245.6	268.3	300.0	-31.7	-0.00048	0.07225	146.2*
0.0325	9.2	22.4	220.0	242.4	300.0	-57.3	-0.00087	0.08470	143.1
0.035	9.2	22.4	191.7	214.1	300.0	-85.9	-0.00131	0.09628	139.7
0.0375	9.1	22.2	172.9	195.1	300.0	-104.9	-0.00160	0.10655	137.4
0.040	9.0	22.0	151.0	173.0	300.0	-127.0	-0.00193	0.11522	134.8
0.0425	9.0	22.0	149.1	171.1	300.0	-128.9	-0.00196	0.12196	134.6
0.045	8.8	21.5	161.6	183.1	300.0	-116.9	-0.00178	0.12674	136.0
0.0475	8.8	21.5	184.9	206.4	300.0	-93.6	-0.00142	0.12974	138.8
0.050	8.7	21.2	207.8	229.0	300.0	-71.0	-0.00108	0.13132	141.5
0.0525	8.6	21.0	222.4	243.4	300.0	-56.6	-0.00086	0.13182**	143.2
								0.13146	

* Maximum shear developed in girder by blast loading.
** Maximum deflection = 0.132 ft < y_m = 0.413 ft, therefore design is satisfactory in bending.

f. Shear Strength and Bond Stress.

$$\text{Maximum shear} = V = 146.2 + \frac{20.0}{2} = 156.2 \text{ kips}$$

$$\text{Shear intensity} = v = V/bjd = \frac{156.2(1{,}000)}{16\left(\frac{7}{8}\right)14} = 797 \text{ psi}$$

$$\text{Allowable } v_y = 0.04 f_c' + 5{,}000p + rf_y \qquad \text{(eq 4.24a)}$$
$$= 0.04(3{,}000) + 5{,}000p + rf_y$$

$$p \text{ of beam web} = \frac{12.48}{16(14)} = 0.0556$$

$$v_y = 120 + 5{,}000(0.0556) + rf_y = 398 + rf_y$$

$$rf_y = r(40{,}000) = 797 - 398 = 399 \text{ psi}$$

$$r = 399/40{,}000 = 0.0102 = \text{ratio of web reinforcement}$$

$$A_s \text{ of stirrups} = 0.010(16)12 = 1.92 \text{ in.}^2 \text{ per ft of beam}$$

Use #5 U stirrups at 3-1/2 in. at ends of span

$$\text{Bond intensity} = u = V/\Sigma ojd = \frac{156.2(1{,}000)}{40\left(\frac{7}{8}\right)14} = 319 \text{ psi}$$

Allowable $u = 0.15 f'_c$ (par. 4-09b)

$= 0.15(3,000) = 450$ psi, OK

g. Design Condition for Longitudinal Roof Girders. Continuous span T-beam, elastic and elasto-plastic action, uniformly distributed mass and roof slab reactions under Zone 1 loading condition, clear span = 11 ft 0 in.; try girder with total depth of 18 in. and width of 12 in.

h. Design Loading. This consists of the sum (V_5) of the following loads:

V_A = reaction from "a" edge of the two-way slab (see table 9.2)

V_2 = reaction from 11.00 ft of corridor slab = 11.00 V_{ave} (see table 9.4)

V_4 = blast load directly applied to girder = $\frac{11(1)144}{1,000} V_3$

$V_5 = V_A + V_2 + V_4$

Table 9.6. Longitudinal Roof Girder Loads (Zone 1 Loading)

t (sec)	V_A (kips)	V_{ave}* (kips/ft)	V_2 (kips)	V_3** (psi)	V_4 (kips)	V_5 (kips)
0	0	0	0	0	0	0
0.0025	6.02	0.84	9.3	3.08	4.44	19.8
0.005	14.90	3.11	34.4	6.16	8.87	58.2
0.0075	24.00	5.68	62.8	9.24	13.30	100.1
0.010	35.10	6.33	70.2	9.84	14.18	119.5
0.0125	42.60	4.84	53.6	9.75	14.05	110.2
0.015	47.10	3.91	43.2	9.68	13.95	104.2
0.0175	51.3	4.94	54.7	9.61	13.82	119.8
0.020	54.5	6.04	66.8	9.55	13.75	135.0†
0.0225	56.5	5.04	55.8	9.48	13.65	126.0
0.025	57.2	2.75	30.6	9.41	13.55	101.4
0.0275	52.1	4.04	44.7	9.35	13.45	110.2
0.030	47.9	5.54	61.3	9.28	13.35	122.6
0.0325	37.0	5.60	62.0	9.20	13.25	112.6
0.035	30.0	4.04	44.8	9.15	13.18	88.0
0.0375	26.8	3.28	36.3	9.10	13.10	76.2
0.040	28.9	2.80	31.0	9.02	13.00	72.9

* V_{ave} = average blast load reaction per one foot strip of corridor slab; see table 9.4.

** V_3 = average Zone 1 overpressure applied directly to girder (fig. 9.16).

† Maximum dynamic load = 135.0 kips.

i. Dynamic Design Factors (Table 6.1B).

Elastic Strain Range:

$$K_{LM} = 0.77, \; V = 0.36R + 0.14P$$

$$R_{1m} = M_{Ps}f_1/L, \; k_1 = EI_1f_3/L^3 \quad \text{(par. 6-20a)}$$

Elasto-plastic Strain Range:

$$K_{LM} = 0.78, \; V = 0.39R + 0.11P$$

$$R_m = \frac{8}{L}(M_{Ps} + M_{Pm}), \; k_{ep} = 384EI_2/5L^3$$

j. Girder Properties. Try same positive and negative steel at center of span and support, respectively, equal to 1.5 percent of web area, with $A_s = 0.015(12)14 = 2.52$ in.^{2} (d = 14 in.). Use 6 #6 bars, $A_s = 2.64$ in.^{2}, $\Sigma o = 14.1$ in., $p = 0.0157$.

$$M_{Ps} = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.70f'_{dc}}\right) \qquad \text{(eq 4.17)}$$

$$= 0.0157(52)\frac{12}{12}(14)^2\left[1 - \frac{0.0157(52)}{1.70(3.9)}\right] = 140 \text{ kip-ft}$$

At center of span, use equation (4.20) for T-beam section.

$b = b' + 8t = 12 + 8(6.5) = 64$ in.

$$a = \frac{A_sf_{dy}}{0.85f'_{dc}b} = \frac{2.64(52)}{0.85(3.9)64} = 0.65 \text{ in.} < 6.5 \text{ in.}$$

$$M_{Pm} = A_sf_{dy}\left(d - \frac{a}{2}\right) = \frac{2.64(52)\left(14 - \frac{0.65}{2}\right)}{12} = 156 \text{ kip-ft}$$

Maximum dynamic load = 135.0 kips

Assume D.L.F. = 1.5 (elasto-plastic behavior)

Estimated R_m required = D.L.F. (maximum dynamic load) = 1.5(135.0) = 203 kips

Weight of slab carried by girder:

$$\text{Portion of two-way slab} = \frac{11\left(\frac{11}{4}\right)\left(\frac{6.5}{12}\right)150}{1{,}000} = 2.5 \text{ kips}$$

$$\text{Portion of one-way slab} = \frac{11\left(\frac{7}{2}\right)\left(\frac{6.5}{12}\right)150}{1{,}000} = 3.1 \text{ kips}$$

$$\text{Stem of girder} = \frac{11(1.5)150}{1{,}000} = 2.5 \text{ kips}$$

Total weight = 8.1 kips

Net R_m furnished (with A_s = 2.64 in.^{2}) $= \frac{8}{L}(M_{Ps} + M_{Pm})$ - dead load

$$= \frac{8}{11}(140 + 156) - 8.1$$

$$= 206.9 \text{ kips}$$

Compute moment of inertia at support I_1:

$$I_g = \frac{bt^3}{12} = \frac{1}{12}(12)18^3 = 5{,}840 \text{ in.}^4$$

For I_t, p = 0.0157, n = 10; from table 1 in Reinforced Concrete Design Handbook, k = 0.425.

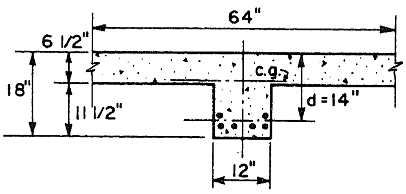
$$I_t = \frac{b(kd)^3}{3} + A_s n(1 - k)^2 d^2$$

$$= \frac{12(18^3)0.425^3}{3} + 2.64(10)(1 - 0.425)^2 (14)^2$$

$$= 1{,}792 + 1{,}710 = 3{,}502 \text{ in.}^4$$

$$I_1 = \frac{1}{2}(I_g + I_t) = (5{,}840 + 3{,}502) = 4{,}761 \text{ in.}^4$$

Compute moment of inertia at center, I_2:



$$\text{Area} = 52(6.5) + 12(18) = 338 + 216 = 554 \text{ in.}^2$$

$$\bar{y} = \frac{338(3.25) + 216(9)}{554} = \frac{1{,}100 + 1{,}946}{554}$$

$$= 5.50 \text{ in.}$$

$$I_g = 338\left(\frac{6.5^2}{12} + 2.25^2\right) + 216\left(\frac{18^2}{12} + 3.5^2\right)$$

$$= 2{,}900 + 8{,}490 = 11{,}390 \text{ in.}^4$$

$$n = 10, \; p = \frac{2.64}{64(14)} = 0.00294, \; k = \sqrt{2np + np^2} - np = 0.174$$

$$I_t = \frac{b(kd)^3}{3} + A_s n(1 - k)^2 d^2 = \frac{64(14)^3(0.174)^3}{3} +$$

$$2.64(10)(1 - 0.174)^2(14)^2 = 308 + 3{,}520 = 3{,}828 \text{ in.}^4$$

$$I_2 = \frac{1}{2}(I_g + I_t) = \frac{1}{2}(11{,}390 + 3{,}828) = 7{,}609 \text{ in.}^4$$

I_1/I_2 = 4,761/7,609 = 0.625

From figure 6.27, f_1 = 13.28

From figure 6.29, f_3 = 485

$$k_1 = \frac{EI_1 f_3}{L^3} = \frac{3(10^3)485(4,761)}{11^3(144)} = 36,000$$

$$R_{1m} = M_{Ps} f_1/L - 8.1 = 140\left(\frac{13.28}{11.07}\right) - 8.1 = 161 \text{ kips}$$

$$y_e = R_{1m}/k_1 = 161/36,000 = 0.00447 \text{ ft}$$

Mass of roof slab and beam = $8.1/32.2 = 0.252$ kip-sec^2/ft

$$T_n = 2\pi\sqrt{K_{LM}m/k_1} = 6.28\sqrt{\frac{0.77(0.252)}{36,000}} = 0.0148 \text{ sec}$$

$$k_{ep} = 384EI_2/5L^2 = \frac{384(3)10^3(7,609)}{5(11.0)^3 144} = 9,150$$

$$y_{ep} = y_e + \frac{R_m - R_{1m}}{k_{ep}} = 0.00447 + \frac{206.9 - 161}{9,150} = 0.00948 \text{ ft}$$

k. Verification of Design and Reaction Determination by Numerical Integration. Use acceleration impulse extrapolation method (par. 5-08d).

$$y_{n+1} = 2y_n - y_{n-1} + \ddot{y}_n(\Delta t)^2 \qquad \text{(eq 5.49)}$$

$$\ddot{y}_n = (P_n - R_n)/K_{LM}m$$

Use $\Delta t = 0.0025$ (approximately 1/10 of t_n, reaction values available for this time interval)

Elastic Strain Range:

$$\frac{(\Delta t)^2}{K_{LM}m} = \frac{0.0025^2}{0.77(0.252)} = 0.0000322$$

$$\ddot{y}_n(\Delta t)^2 = 0.0000322\ (P_n - R_n)$$

$$R_n = k_1 y_n = 36,000y_n,\ (y_n \le 0.00447 = y_e)$$

P_n = obtained from Design Loading (par. 9-10b)

$V = 0.36R + 0.14P$

Elasto-plastic Strain Range:

$$\frac{(\Delta t)^2}{K_{LM}m} = \frac{0.0025^2}{0.78(0.252)} = 0.0000318$$

$$R_n = R_{lm} + k_{ep}(y_n - y_e) = 161 + 9{,}150\,(y_n - y_e)$$

$$= 120 + 9{,}150\,y_n,\ (y_e = 0.00447 \le y_n \le 0.00948 = y_{ep})$$

P_n = same as above

$V = 0.39R + 0.11P$

Table 9.7. Determination of Maximum Deflection and Dynamic Reactions for Longitudinal Roof Girder (Zone 1 Loading)

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	Strain Range	V (kips)
0	0	0	19.8/6	0.00011	0	e	0
0.0025	19.8	4.0	15.8	0.00051	0.00011	e	4.2
0.005	58.2	26.3	31.9	0.00103	0.00073	e	17.7
0.0075	100.1	85.7	14.4	0.00046	0.00238	e	44.9
0.010	119.5	161.1	-41.6	-0.00133	0.00449	e	74.7
0.0125	110.2	168.3	-58.1	-0.00185	0.00527	e-p	77.8
0.015	104.2	129.5*	-25.3	-0.00082	0.00420	e	61.2
0.0175	119.8	61.4*	+58.4	+0.00188	0.00231	e	38.9
0.020	135.0	61.2*	+73.8	+0.00238	0.00230	e	40.9
0.0225	126.0	146.5*	-20.5	-0.00066	0.00467	e	70.4
0.025	101.4	178.4	-77.0	-0.00245	0.00638**	e-p	80.8†
0.0275	110.2	151.5††	-41.3	-0.00133	0.00564	e	74.5
0.030	122.6	77.4††	45.2	+0.00146	0.00357	e	47.4
0.0325	112.6	55.4††	57.2	+0.00184	0.00296	e	37.4
0.035	88.0	99.6††	-11.6	-0.00037	0.00419	e	51.1
0.0375	76.2	130.7††	-54.5	-0.00176	0.00505	e	61.7
					0.00415		

* $R_n = 168.3 - (0.00527 - y_n)36{,}000 = 36{,}000y_n - 21.7$.

** Maximum deflection = 0.0064 ft. $y_{ep} = 0.00884 > 0.0064 > 0.0047 = y_e$. Therefore design is satisfactory in bending.

† Maximum dynamic reaction = 80.8 kips.

†† $R_n = 178.4 - (0.00638 - y_n)36{,}000 = 36{,}000y_n - 51.3$.

1. <u>Shear Strength and Bond Stress.</u>

$$\text{Maximum shear} = V = 80.8 + \frac{8.1}{2} = 84.8 \text{ kips}$$

$$\text{Shear intensity} = v = V/bjd = \frac{84.8(1{,}000)}{12(7/8)\,14} = 576 \text{ psi}$$

$$\text{Allowable } v_y = 0.04f'_c + 5{,}000p + rf_y \qquad \text{(eq 4.24)}$$

$$p \text{ beam web} = \frac{2.64}{14(12)} = 0.0157$$

$v_y = 120 + 5{,}000(0.0157) + rf_y = 198 + rf_y$

$rf_y = r(40{,}000) = 576 - 198 = 378$ psi

$r = 378/40{,}000 = 0.0095$ = ratio of web reinforcement

A_s of stirrups $= 0.0095(12)12 = 1.37$ in.^{2} per ft of beam

Use #5 at 5 in. at each end of beam.

$$\text{Bond intensity} = u = V/\Sigma ojd = \frac{84.8(1{,}000)}{14.1(7/8)14} = 490 \text{ psi}$$

Allowable $u = 0.15f'_c$ (par. 4-09b)

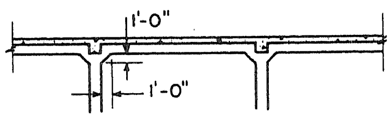
$= 0.15(3{,}000) = 450 \text{ psi} < 490 \text{ psi}$

Make another selection of steel on basis of greater Σo

Use 9 #5 bars at support, $A_s = 2.79$, $\Sigma o = 17.6$

$$\text{Bond intensity } u = V/\Sigma ojd = \frac{84.8(1{,}000)}{17.6(7/8)14} = 393 \text{ psi} < 450 \text{ psi} \quad \text{OK}$$

m. Haunch on Longitudinal Girders. The design presented for longitudinal girders results in a relatively large shear stress at the intersection of longitudinal and transverse girders. The width of the transverse girder (16 in.) framing into the longitudinal girder will greatly increase this shear stress at the top of the column unless the column dimension in this direction is a minimum of 16 in. In order not to place this lower limit on column size, a haunch will be used at each end of the longitudinal girders to receive the 16-in.-wide transverse girders framing into them. Haunches 12 in. along column and 12 in. along longitudinal girders will be used. The actual depth of the section will be considered as resisting shear in accordance with ACI Code 702(d) in reference [9].



9-11 COLUMN AND COLUMN FOOTING DESIGN. a. Column Design Conditions. Axially loaded tied columns will be designed in accordance with the principles stated in paragraph 4-11.

b. Design Loading. Using girder reactions from tables 9.5 and 9.7, a maximum column load G_4 is determined by summing the loads shown in table 9.8. The total load on the column is the sum of the dynamic reactions of the girders that it supports under Zone 1 roof loading with the blast wave traveling parallel to the longitudinal axis of the structure.

The loads from the various girders must be summed, taking care to provide the proper time phasing.

Table 9.8. Column Loads (Zone 1 Loading)

G_1 = longitudinal girder reaction at time, t_n (table 9.7)
G_2 = transverse girder reaction at time, t_n (table 9.7)
G_3 = longitudinal girder reaction at time, $t_n - t_{n - 0.0075}$ (table 9.7)
$G_4 = G_1 + G_2 + G_3$

t (sec)	G_1 (kips)	G_2 (kips)	G_3 (kips)	G_4 (kips)
0	0	0	----	0
0.0025	4.2	1.39	----	5.6
0.005	17.7	3.94	----	21.6
0.0075	44.9	9.76	0	54.7
0.010	74.7	16.78	4.2	95.7
0.0125	77.8	26.00	17.7	121.5
0.015	61.2	37.20	44.9	143.3
0.0175	38.9	52.20	74.7	165.8
0.020	40.9	69.40	77.8	188.1
0.0225	70.4	88.50	61.2	220.1
0.025	80.8	110.0	38.9	229.7
0.0275	74.5	131.0	40.9	246.4
0.030	47.4	146.2	70.4	264.0
0.0325	37.4	143.1	80.8	261.3
0.035	51.1	139.7	74.5	265.3*
0.0375	61.7	137.4	47.4	246.5

* Maximum dynamic column reaction = 265.3 kips.

Static column reaction = $2\left(\frac{8.1}{2}\right) + \left(\frac{20.0}{2}\right) = 18.1$ kips
(weight of column is neglected)

Total maximum column reaction = 265.3 + 18.1 = 283.4 kips

Try column 12 in. by 12 in.

$$P_p = 0.8\left[0.9(0.85)\,f'_{dc}A_c + A_s f_{dy}\right] \qquad \text{(eq 4.27)}$$

Load taken by concrete = 0.8(0.9)0.85(3.9)144 = 341 kips

341 kips > 283.4 kips. Use minimum steel ($p \approx 0.01$)

$A_s = 0.01(144 \text{ in.}^2) = 1.44 \text{ in.}^2$ Use four #6 bars. $A_s = 1.76 \text{ in.}^2$

Column capacity = 341 + 0.8(1.76)52 = 414.2 kips > 283.4 kips. OK

Use column ties in accordance with ACI standard procedure.

c. Soil Pressure Investigation.

Try 3-ft-6-in. by 3-ft-6-in. by 1-ft-6-in. column footing

Maximum column load on footing = 283.4 kips

Column weight = 0.15(1)1(10) = 1.5 kips

6-in. floor slab over footing = 0.15(3.5)3.5(0.5) = 0.9

6-in. fill over footing = 0.12(3.5)3.5(0.5) = 0.7

18-in. footing = 0.15(3.5)3.5(1.5) = 2.8

Total force on foundation = 289.3 kips

Maximum static and dynamic bearing pressure = 289.3/3.5(3.5)

$= 23.6 \text{ kips/ft}^2 < 30 \text{ kips/ft}^2$

The maximum bearing pressure is less than the ultimate load-bearing capacity of the foundation material, therefore the 3-ft-6-in. by 3-ft-6-in. by 1-ft-6-in. footing is satisfactory.

d. Column Footing Design.

Footing moment at face of column $= \dfrac{23.6(1.25)^2}{2} = 18.5$ kip-ft/ft

Footing shear at face of column = 23.6(1.25) = 29.5 kips/ft

Determine minimum depth of footing with 1.5 percent steel. From paragraph 9-09e using equation (4.17)

$M_p = 0.688d^2$ kip-ft/ft

$18.5 = 0.688d^2$, $d = 5.2$ in.

Try 14-in. deep footing, d = 10 in., try minimum p = 0.006

$$M_p = pf'_{dy}bd^2 \left(1 - \frac{pf'_d}{1.70\ f'_{dc}}\right)$$

$$= 0.006(52)\frac{12}{12}(10^2) \left[1 - \frac{0.006(52)}{1.70(3.9)}\right] = 29.8 \text{ kip-ft/ft} > 18.5$$

Allowable bond stress $= 0.15f'_c$ (par. 4-09b)

$= 450$ psi

Required $\Sigma o = \dfrac{V}{ujd} = \dfrac{29.5(1{,}000)}{450(12)(7/8)10} = 0.62$ in./ft

Required $A_s = pbd = 0.006(12)10 = 0.72 \text{ in.}^2\text{/ft}$

Use #6 bars at 6 in. each way.

Furnished $A = 0.88 \text{ in.}^2$, $\Sigma o = 4.7$ in./ft

e. Shear Strength. Total shear at distance "d" from faces of column:

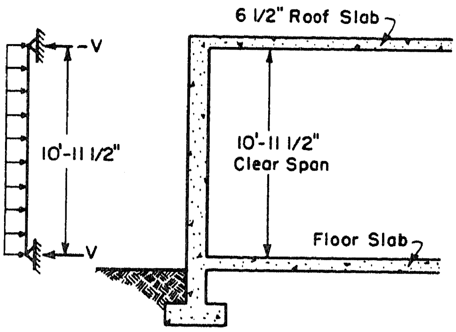
$$23.6\left[(3.5)^2 - (1 + 1.67)^2\right] = 121 \text{ kips} = V$$

$$\text{Shear intensity} = v = \frac{V}{bjd} = \frac{121(1{,}000)}{2.67(4)(7/8)10(12)} = 108 \text{ psi}$$

Allowable shear intensity (eq 4.24), $v_y = 0.04f'_c + 5{,}000p + rf_y$
$= 150 \text{ psi} > 108 \text{ psi}$

Therefore footing 3-ft-6-in. by 3-ft-6-in. by 1-ft-2-in. is OK.

9-12 WALL SLAB DESIGN. a. Design Conditions. Simple span action will be used in design of walls for blast loads normal to the walls. The lack of moment restraint exists because of the relatively thin roof slab at top of span and flexibility of earth reaction at bottom of span. The front and rear walls will also act as deep beams because of overturning action.



That portion of the design cannot be presented until after the dynamic overturning and sliding analysis is performed.

b. Design for Blast Loads Normal to Front Wall. Simple span, elastic and plastic action, uniformly distributed mass and loading, clear span = 10 ft 11-1/2 in. $x_m/x_e = \alpha\beta = 6$ (par. 6-26), minimum thickness for proper steel placement = 10 in.

c. Design Loading. Use figure 9.19, average front wall overpressure-time curve.

d. Dynamic Design Factors (Table 6.1A).

Elastic Strain Range:

$$K_{LM} = 0.78,\ R_m = 8M_p/L,\ k = 384EI/5L^3$$

$$V = 0.39R + 0.11P$$

Plastic Strain Range:

$$K_{LM} = 0.66,\ V = 0.38R_m + 0.12P$$

e. Slab Properties (per foot of width).

Try 10-in. wall (minimum thickness) with p about 0.015.

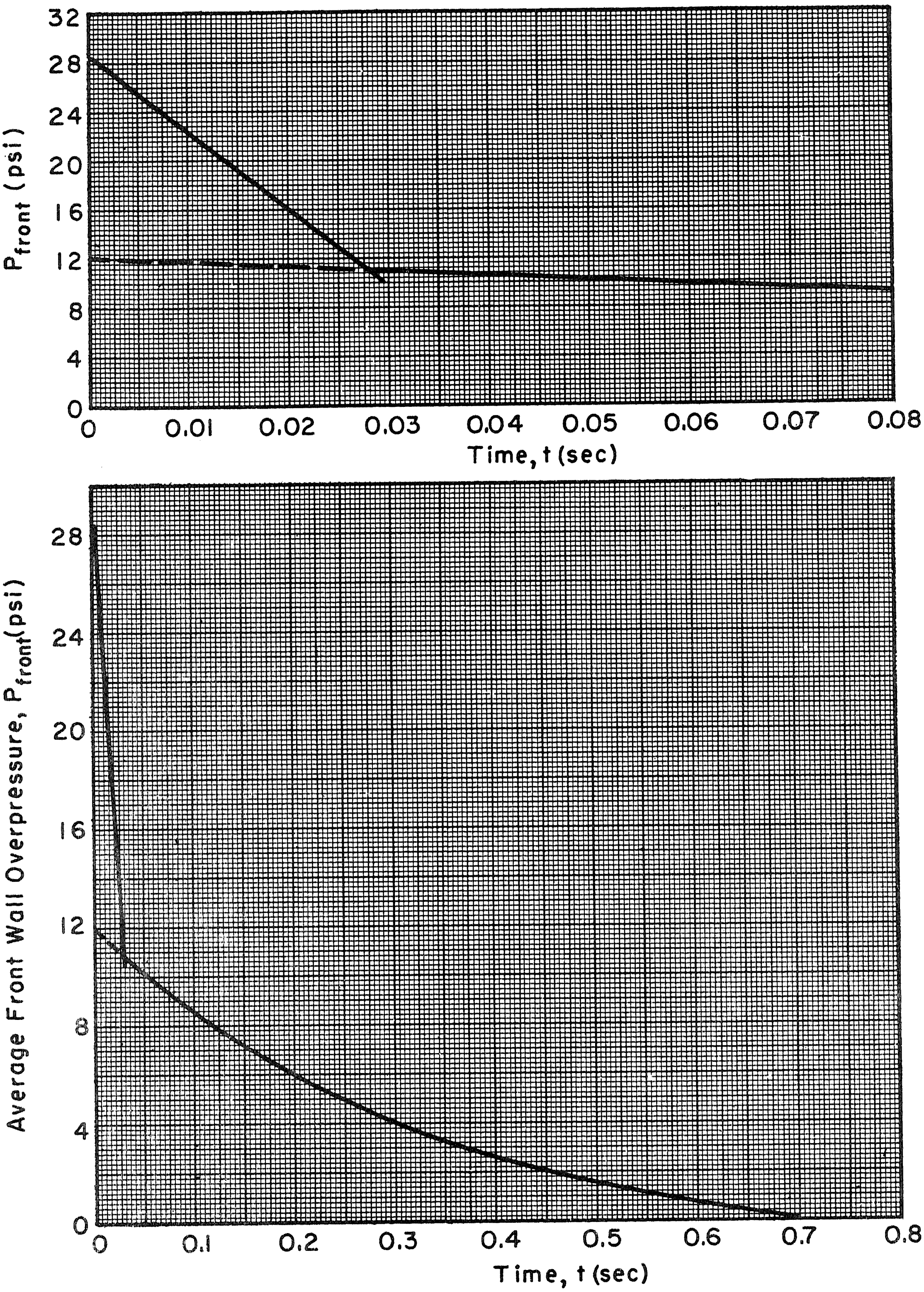


Figure 9.19. Average front wall overpressure-time curve

$d = 8.5$ in. at center of span

Try #6 at 3-1/2 in., $A_s = 1.51$ in.^{2}/ft, $\Sigma o = 8.1$ in./ft, $p = 0.0148$

$$M_p = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.70f_{dc}}\right)$$

$$= 0.0148(52)\frac{12}{12}(8.5^2)\left[1 - \frac{0.0148(52)}{1.70(3.9)}\right] = 49.1 \text{ kip-ft}$$

$$R_m = \frac{8M_p}{L} = \frac{8(49.1)}{10.96} = 35.9 \text{ kips}$$

$$m = \frac{(10/12)150(10.96)}{1,000(32.2)} = 0.0425 \text{ kip-sec}^2\text{/ft}$$

$$I_g = \frac{bt^3}{12} = \frac{12(10^3)}{12} = 1,000 \text{ in.}^4$$

For $n = 10$, $p = 0.0148$, $k = 0.382$, from table 1 of Reinforced Concrete Handbook.

$$I_t = \frac{1}{3}b(dk)^3 + nA_s\left[(1 - k)d\right]^2 = \frac{12(8.5)^3(0.382)^3}{3} +$$

$$10(1.51)(1 - 0.382)^2(8.5)^2 = 137 + 416 = 553 \text{ in.}^4$$

$$I_a = \frac{1}{2}(I_g + I_t) = \frac{1}{2}(1,000 + 553) = 776 \text{ in.}^4$$

$$k = \frac{384EI}{5L^3} = \frac{384(3)10^3(776)}{5(10.96)^3 144} = 946 \text{ kips/ft}$$

$$T_n = 2\pi\sqrt{K_{LM}m/k} = 6.28\sqrt{\frac{0.78(0.0425)}{946}} = 0.037 \text{ sec}$$

f. Verification of Design and Reaction Determination by Numerical Integration. Use acceleration impulse extrapolation method (par. 5-08d).

$$y_{n+1} = 2y_n - y_{n-1} + \ddot{y}_n(\Delta t)^2 \qquad \text{(eq 5.49)}$$

$$\ddot{y}_n = (P_n - R_n)/K_{LM}m$$

$\Delta t = 0.0025$ (approximately 1/10 T_n, same value used throughout design)

Elastic Strain Range:

$$\frac{(\Delta t)^2}{K_{LM}m} = \frac{0.0025^2}{0.78(0.0425)} = 0.0001886$$

$$\ddot{y}_n(\Delta t)^2 = 0.0001886\,(P_n - R_n) \text{ ft}$$

$$P_n = \frac{10.96(144)\bar{P}_{front}}{1{,}000} = 1.578\bar{P}_{front} \text{ kips}$$

$\bar{P}_{front}$ is obtained from figure 9.19.

$V = 0.39R + 0.11P$

$R_n = ky_n = 946y_n$ kips, $(y_n \le y_e = 35.9/946 = 0.0379$ ft)

Plastic Strain Range:

$$\frac{(\Delta t)^2}{K_{LM}m} = \frac{0.0025^2}{0.66(0.0425)} = 0.000223$$

$\ddot{y}_n(\Delta t)^2 = 0.000223\ (P_n - R_m)$ ft

P_n is same as above for elastic case.

$R_m = 35.9$ kips

$V = 0.38R_m + 0.12P$

$x_m = 6x_e = 6(0.0379) = 0.228$ ft

When $x_{n+1} < x_n$, $R_n = R_m - 946\ (y_{max} - y_n)$

Table 9.9. Determination of Maximum Deflection and Dynamic Reactions for Front Wall Slab

t (sec)	$\bar{P}_{front}$ (psi)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	Strain Range	V (kips)
0	28 5	45.0	0	45.0/2	0.00425	0	e	4.96
0.0025	26.9	42.5	4.0	38.5	0.00726	0.00425	e	6.24
0.005	25.3	39.9	14.9	25.0	0.00471	0.01576	e	10.21
0.0075	23.7	37.4	30.3	7.1	0.00134	0.03198	e	15.92
0.010	22.1	34.8	35.9	-1.1	-0.00025	0.04954	p	17.80*
0.0125	20.5	32.4	35.9	-3.5	-0.00078	0.06685	p	17.51
0.015	18.9	29.8	35.9	-6.1	-0.00136	0.08338	p	17.20
0.0175	17.3	27.3	35.9	-8.6	-0.00192	0.09855	p	16.90
0.020	15.8	25.0	35.9	-10.9	-0.00243	0.11180	p	16.62
0.0225	14.2	22.4	35.9	-13.5	-0.00301	0.12262	p	16.31
0.025	12.6	19.9	35.9	-16.0	-0.00357	0.13043	p	16.01
0.0275	11.1	17.5	35.9	-18.4	-0.00410	0.13467	p	15.72
0.030	10.8	17.0	35.9	-18.9	-0.00421	0.13481**	p	15.66
0.0325	10.7	16.9	32.0†	-15.1	-0.00285	0.13074	e	14.36
0.035	10.6	16.7	25.5†	-8.8	-0.00196	0.12382	e	11.78
0.0375	10.5	16.6	17.3†	-0.7	-0.00013	0.11494	e	8.57
0.040	10.4	16.4	8.5†	+7.9	+0.00149	0.10593	e	5.13
0.0425	10.3	16.2	1.5†	+14.7	+0.00278	0.09841	e	2.37
0.045	10.2	16.1	-2.9	+19.0	+0.00358	0.09367	e	0.64
0.0475	10.1	15.9	-4.0			0.09251	e	0.19

* Maximum dynamic reaction = 17.8 kips.

** Maximum deflection = 0.135 ft $< 0.228 = x_m$. Therefore design is satisfactory in bending.

† $R_n = 35.9 - (0.13481 - y_n)946 = 946y_n - 91.5$.

g. Shear Strength and Bond Stress.

Maximum shear = 17.8 kips/ft = V

Maximum shear intensity = $v = V/bjd = \frac{17.8(1{,}000)}{12(7/8)8.5} = 199$ psi

Allowable $v_y = 0.04f'_c + 5{,}000p + rf_y$ (eq 4.24a)

$= 0.04(3{,}000) + 5{,}000(0.0148) = 120 + 74 = 194 \text{ psi} < 199$

The allowable shear stress is exceeded by 2.5 percent; however, this is tolerable and no stirrups are used.

Maximum bond intensity = $u = V/\Sigma ojd = \frac{17.8(1{,}000)}{8.1(7/8)8.5} = 296$ psi

Allowable $u = 0.15f'_c$ (par. 4-09b)

$= 0.15(3{,}000) = 450 \text{ psi} > 296$

The bond stress intensity is satisfactory.

h. Back Wall Slab Reactions. Since it is necessary to include the effect of blast on back of structure in the dynamic overturning and sliding analysis to be performed later, the back wall slab reactions will be obtained in a manner similar to that used for front wall slab reactions.

i. Design Loading. See average back wall overpressure-time curve (fig. 9.20).

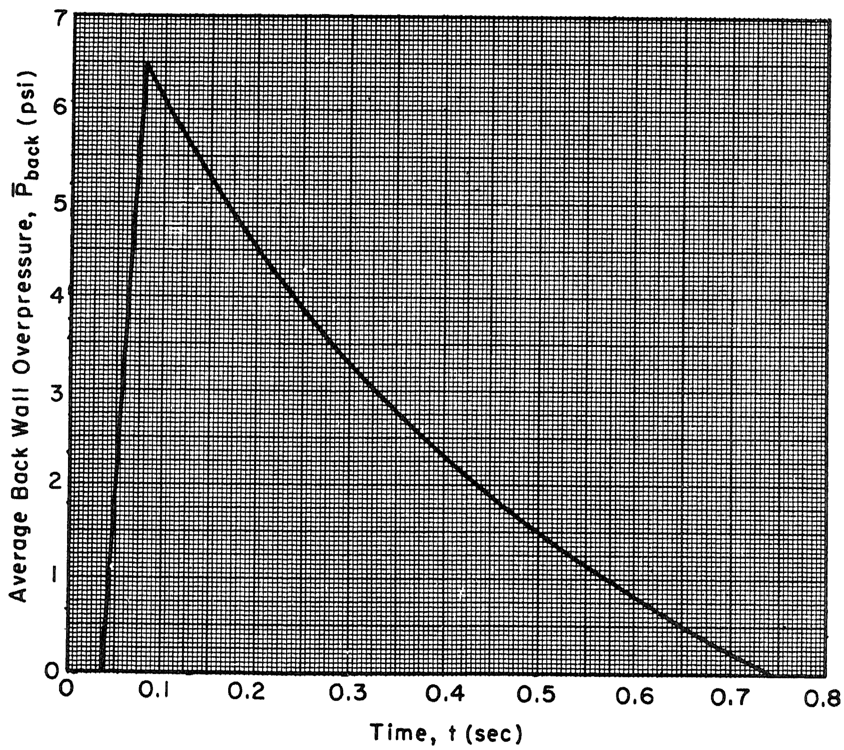


Figure 9.20. Average back wall overpressure-time curve

j. Design Factors and Wall Properties. Same as front wall slab.

k. Reaction Determination by Numerical Integration. Using the acceleration impulse extrapolation method (par. 5-08d) and the same resistance functions as in f. above, the dynamic reactions can be obtained for the back wall slab.

Table 9.10. Determination of Maximum Deflection and Dynamic Reactions for Back Wall Slab

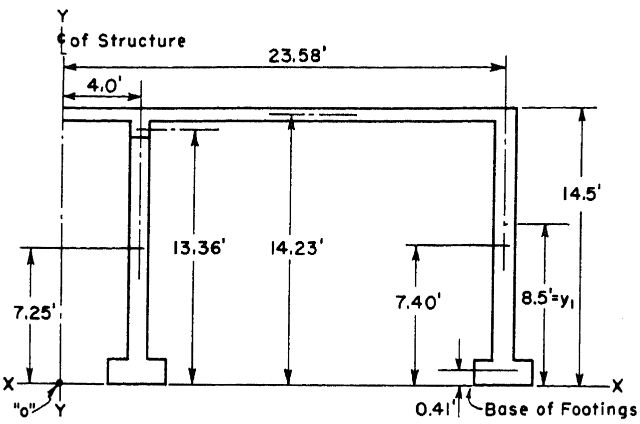
t (sec)	$\bar{P}_{back}$ (psi)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	Strain Range	V (kips)
0.030	0	0	0					
0.0325	0	0	0					
0.035	0.20	0.316	0	0.316/2	0.0000298	0	e	0.0348
0.0375	0.53	0.835	0.028	0.803	0.0001518	0.0000298	e	0.1029
0.040	0.85	1.340	0.200	1.140	0.0002150	0.0002114	e	0.228
0.0425	1.18	1.86	0.575	1.285	0.0002420	0.0006090	e	0.435
0.045	1.60	2.52	1.180	1.340	0.0002530	0.0012486	e	0.738
0.0475	1.93	3.04	2.030	1.010	0.0001910	0.0021412	e	1.129
0.050	2.30	3.62	3.060	0.560	0.0001058	0.0032248	e	1.588
						0.0044132		

9-13 RIGID BODY OVERTURNING AND SLIDING ANALYSIS. In order to perform the preliminary shear wall analysis later in this example, it is necessary first to consider the building as a rigid body, sliding and rotating about the longitudinal axis of the footing. Using the principles presented in paragraph 9-06a, it is necessary to compute $\ddot{x}_o$ and α_o and the corresponding displacement and rotation, x_o and θ_o, respectively, by means of a concurrent numerical integration of equations (9.4) and (9.5). From these values, the rigid body acceleration of the top of the shear wall can be obtained and used in the shear wall design.

$$\alpha_o = \frac{M_o - F_o\bar{y}}{I_o - m\bar{y}^2} \qquad \text{(eq 9.4)}$$

$$\ddot{x}_o = \frac{F_o}{m} - \alpha_o\bar{y} \qquad \text{(eq 9.5)}$$

a. Mass Moment of Inertia, I_o This consists of the mass moment of inertia of all of the elements about the axis of rotation of the structure.



Because of the tendency for certain parts of the structure such as the earth, floor slab, column footings, etc., to translate but not rotate, these elements will have an inertial component in the horizontal direction only. All other portions of the structure will have inertial components in both the vertical and horizontal directions. The centroid and moment of inertia computations are presented in tables 9.11, 9.12, and 9.13. The dimensions used in these computations are shown in the sketch.

Table 9.11. Computation of Mass and Location of Centroid of Building

Element of Building	Dimensions	Volume (cu ft)	Weight (kips)	Mass (m) $\left(\frac{\text{kip-sec}^2}{\text{ft}}\right)$	$\bar{y}$* (ft)	$m\bar{y}$	Ratio of Rotating Mass to Total Mass		Rotating Mass $\left(\frac{\text{kip-sec}^2}{\text{ft}}\right)$
Front wall	0.83(13.13)156.8	1,715	257	7.96	7.40	59.0		1.0	7.96
Rear wall	0.83(13.13)156.8	1,715	257	7.96	7.40	59.0		1.0	7.96
End walls	2 [0.83(46.34)13.13] - 84	926	139	4.32	7.40	32.2		1.0	4.32
Shear walls	4 [0.83(46.34)13.13] - 52.5	1,810	271	8.40	7.40	62.2		1.0	8.40
Roof slab	0.54(48.0)156.8	4,060	608	18.85	14.23	269.0	$1 - \frac{2[28(12) + 1.5(28)24]}{48(157)}$	0.64	12.09
Transverse girders	8(0.96)133(39.0)	396	59.4	1.84	13.36	24.5	One-half length only	0.50	0.92
Longitudinal girders	2(0.96)1.0(154.8)	298	44.6	1.38	13.36	18.45	8/13	0.62	0.85
Columns	(36)1.0(1.0)12.17	438	65.8	2.04	7.25	14.90	Interior columns only (20)	0.55	1.13
Floor	0.5(46.34)155.17	3,600	540.0	17.00	2.75	46.70		0	-----
Exterior wall footings	2(0.83)2.5(201.0)	835	125.0	3.90	0.43	1.68		1.0	3.90
Isolated footings	16(4.0)4.0(1.5)	384	57.6	1.78	0.50	0.89		0	-----
Attached footings	20(1.67)2.67(0.83)	74	1.1	0.34	0.43	0.15		1.0	0.34
Struts	8(1.0)1.0(33.42)	267	40.0	1.24	0.50	0.62		0	-----
Shear wall footings	4(0.83)2.5(45.48)	377	56.5	1.75	0.41	0.75		1.0	1.75
Earth	6.5(23.17)46.33(1.67) 6.5(874)0.83	16,390	1,639	36.20 14.60	1.67 0.43	60.50 6.27		0	-----
			4,151.0	129.56		656.81			49.62

Note: Net $\bar{y}$ = 656.81/129.56 = 5.06 ft.
* $\bar{y}$ is vertical distance of mass center of element above point of rotation "O."

Table 9.12. Computation of Moment of Inertia, I_{YY}

Element	$\frac{1}{12}md^2$		$m\bar{x}^2$		I_{YY} (kip-sec^2/ft)
Front wall	$0.0833(7.96)0.83^2$	0.46	$7.96(23.58)^2$	4,420	4,420
Rear wall	$0.0833(7.96)0.83^2$	0.46	$7.96(23.58)^2$	4,420	4,420
End wall	$0.0833(4.32)(46.34)^2$	775.0			775
Shear walls	$0.0833(8.40)(19.66)^2$	270.0	$8.40(17.38)^2$	2,530	2,800
Roof slab	$0.083\left[18.85(48.0)^2 - 6.76(28.0)^2\right]$	3,180.0			3,180
Transverse girders	$0.0833(0.92)19.66^2$	29.6	$0.92(17.38)^2$	2,760	2,790
Longitudinal girders	$0.0833(0.53)1.0^2$	0.04	$0.53(4.0)^2$	8	8
Columns	$0.0833(0.91)1.0^2$	0.08	$0.91(20.0)^2$	364	364
Floor					
Exterior wall footing (longitudinal only)	$0.0833(3.07)2.50^2$	1.59	$3.07(23.58)^2$	1,700	1,702
Isolated footings					
Attached footing	$0.0833(0.34)1.67^2$	0.07	$0.34(20.0)^2$	136	136
Struts	$0.0833(0.62)8.35^2$	3.60	$0.62(17.38)^2$	187	191
Shear wall footing	$0.0833(2.58)44.75^2$	432.0			432
Earth					
					21,118

Table 9.13. Computation of Moment of Inertia, I_{XX}

Element	$\frac{1}{12}md^2$		$m\bar{y}^2$		I_{XX} (kip-sec^2/ft)
Front wall	$0.083(7.96)13.13^2$	114	$7.96(7.40)^2$	435	549
Rear wall	$0.083(7.96)13.13^2$	114	$7.96(7.40)^2$	435	549
End wall	$0.083(4.32)13.13^2$	62	$4.32(7.40)^2$	236	298
Shear walls	$0.083(8.40)13.13^2$	121	$8.40(7.40)^2$	458	579
Roof slab	$0.083(18.85)0.54^2$	0.5	$18.85(14.23)^2$	3,830	3,830
Transverse girders	$0.083(1.84)0.96^2$	0.1	$1.84(13.36)^2$	329	329
Longitudinal girders	$0.083(1.38)0.96^2$	0.1	$1.38(13.36)^2$	247	247
Columns	$0.083(2.04)12.17^2$	25	$2.04(7.25)^2$	107	132
Floor	$0.083(17.00)0.50^2$	0.4	$17.00(2.75)^2$	128	128
Exterior wall footing	$0.083(3.90)0.83^2$	0.2	$3.90(0.41)^2$	0.6	1
Isolated footing	$0.083(1.78)1.50^2$	0.3	$1.78(0.41)^2$	0.2	1
Attached footing	$0.083(0.34)0.83^2$		$0.34(0.41)^2$		
Struts	$0.083(1.24)1.0^2$	0.1	$1.24(0.50)^2$	0.3	
Shear wall footing	$0.083(2.58)0.83^2$	0.2	$2.58(0.50)^2$	0.6	1
Earth	$0.083(36.20)1.67^2$	83	$36.20(1.67)^2$	101.0	134
Earth	$0.083(14.60)0.83^2$	8.3	$14.60(0.41)^2$	2.4	11
					6,839

$$I_o = I_{YY} + I_{XX} = 21{,}118 + 6{,}839 = 27{,}957 \text{ kip-sec}^2/\text{ft}$$

b. Ground Foundation Interaction.

$$B = \frac{\pi E L^2}{4(1 - \nu)^2} \qquad \text{(eq 4.59)}$$

This equation is for a solid foundation slab and the value of B thus obtained must be multiplied by the ratio of I_{net}/I_{gross} since the foundation consists essentially of strip footings. See paragraph 4-15d.

$$\frac{I_{net}}{I_{gross}} = \frac{\frac{1}{12} bd^3 - \frac{1}{12} b'd'^3}{\frac{1}{12} bd^3} = 1 - \frac{b'd'^3}{bd^3}$$

From figures 9.12 and 9.13,

$b = 158.5$ ft

$d = 49.75$ ft $= 2L$

$b' = 143.5$ ft

$d' = 44.75$ ft

$$\text{Ratio } I_{net}/I_{gross} = 1 - \frac{143.5(44.75)^3}{158.5(49.75)^3} = 0.340$$

For a solid slab,

$$B' = \frac{\pi}{4} \; \frac{40}{1 - 0.333^2} \left[\frac{49.75(12)}{2}\right]^2 = 31.5(10)^5 \text{ ft-kips/ft/radian}$$

Modified for specific example,

$$B = 31.5(10)^5 158.5(0.340) = 17.0(10^7) \text{ ft-kips/radian}$$

c. Determination of Passive Resistance of Earth, F_p. Since earth between front and rear wall will be considered to move with the structure, the rear wall only will offer passive resistance.

$$F_p = 0.5(\gamma)H^2(K_{P\phi}) \qquad \text{(eq 4.58)}$$

$$= 0.5(0.100)2.5^2(10) = 3.12 \text{ kips/ft}$$

3.12 kips/ft (156.8) = 492 kips = total passive pressure

d. Determination of Blast Forces on Front Wall F_f, Back Wall F_b, and Roof V.

$$F_f = 12.0\ (156.83)(144/1{,}000)\overline{P}_{front} = 270\overline{P}_{front} \text{ kips}$$

$F_b = 270\ \overline{P}_{back}$ kips

$F_f - F_b = 270(\overline{P}_{front} - \overline{P}_{back})$ kips

$\overline{P}_{front} - \overline{P}_{back}$) is obtained from figure 9.21.

V = vertical blast load on roof and portion of footings exposed to blast

Blast load on roof $(V_r) = 48.0(156.8)\ \frac{144}{1,000}\ \overline{P}_{roof} = 1,080\ \overline{P}_{roof}$ kips

$\overline{P}_{roof}$ is obtained from figure 9.18.

Blast load on footing $(V_P) = 0.83(158.5)\frac{144}{1,000}(\overline{P}_{front} + \overline{P}_{back})$
$= 19.1\ (\overline{P}_{front} + \overline{P}_{back})$. (Exposed end wall footing not included.)

$\overline{P}_{front}$ and $\overline{P}_{back}$ are obtained from figures 9.19 and 9.20, respectively.

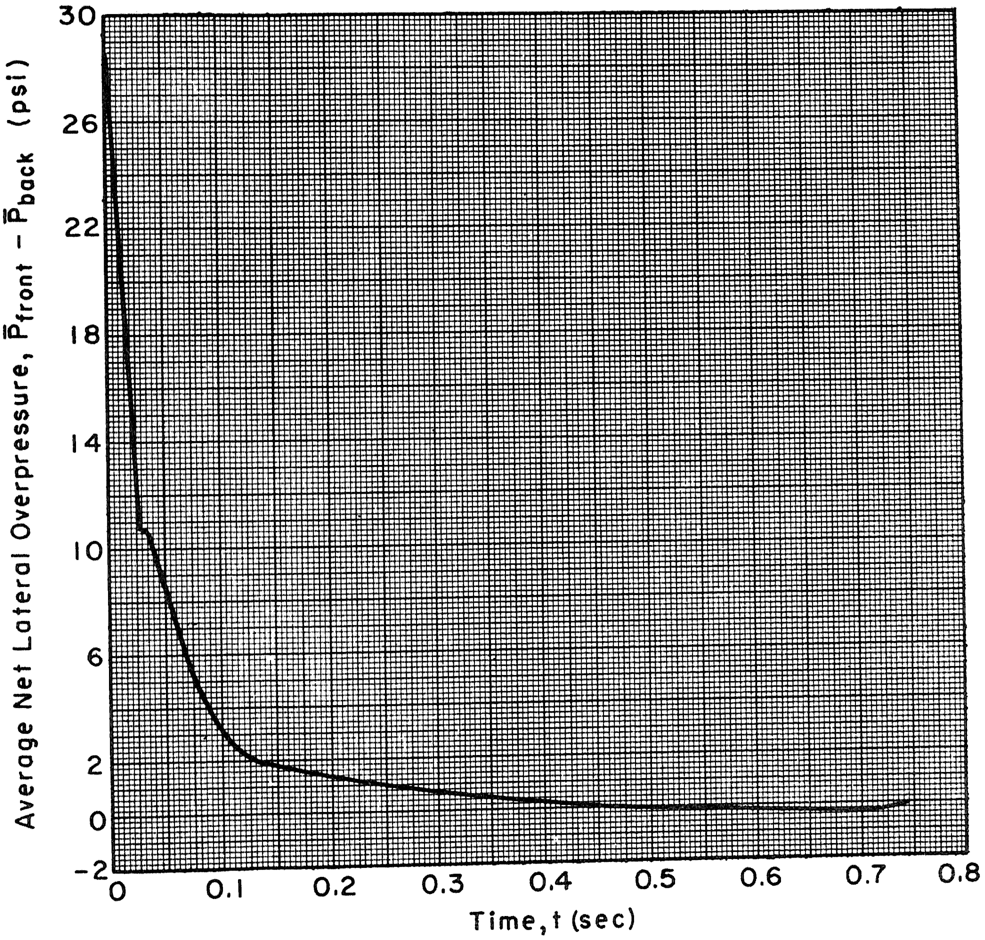


Figure 9.21. Average net lateral overpressure versus time

Table 9.14. (F_f - F_b) and V Values for Rigid Body Analysis

t (sec)	$\bar{P}_{front} - \bar{P}_{back}$ (psi)	$F_f - F_b$ (kips)	$\bar{P}_{front} + \bar{P}_{back}$ (psi)	V_p (kips)	$\bar{P}_{roof}$ (psi)	V_r (kips)	V (kips)
0	28.5	7,700	28.5	545	0	0	545
0.0025	26.9	7,260	26.9	514	0.60	647	1,161
0.005	25.2	6,800	25.2	482	1.20	1,290	1,772
0.0075	23.7	6,400	23.7	452	1.80	1,945	2,397
0.010	22.1	5,960	22.1	422	2.50	2,700	3,122
0.0125	20.5	5,540	20.5	391	3.10	3,350	3,741
0.015	18.9	5,100	18.9	361	3.65	3,940	4,301
0.0175	17.3	4,720	17.3	330	4.30	4,650	4,980
0.020	15.7	4,240	15.7	300	5.00	5,400	5,700
0.0225	14.1	3,800	14.1	269	5.60	6,040	6,369
0.025	13.6	3,670	13.6	260	6.15	6,640	6,900
0.0275	11.0	2,970	11.0	210	6.75	7,280	7,490
0.030	10.8	2,920	10.8	206	7.45	8,040	8,246
0.0325	10.7	2,880	10.7	204	8.05	8,700	8,964
0.035	10.6	2,860	10.6	202	8.65	9,340	9,542
0.0375	9.8	2,640	10.5	200	8.62	9,300	9,500
0.040	9.5	2,560	10.4	198	8.45	9,120	9,318
0.0425	9.1	2,460	10.3	197	8.35	9,000	9,197
0.045	8.6	2,320	10.2	195	8.30	8,950	9,045
0.0475	8.2	2,220	10.1	193	8.20	8,850	9,043
0.0500	7.8	2,100	10.0	191	8.09	8,720	8,911
0.0525	7.4	2,000	11.4	218	8.00	8,640	8,858
0.055	6.8	1,835	12.9	246	7.90	8,540	8,786
0.0575	6.3	1,700	13.2	252	7.82	8,430	8,682
0.060	5.9	1,590	13.4	256	7.75	8,360	8,616

e. Dynamic Analysis.

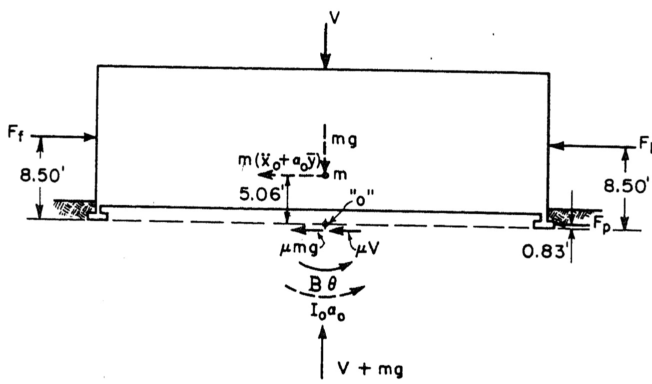


Table 9.15. Simultaneous Rigid Body Overturning and Sliding Analysis. Part I – Overturning

t (sec)	$F_f - F_b$ (kips)	$F_f y_f - F_b y_b$ (ft-kips)	$F_p y_o$ (ft-kips)	$B\theta$ (ft-kips)	M_o (ft-kips)	μmg	μV	F_o (kips)	$M_o - F_o\bar{y}$ (ft-kips)	α (radians/sec^2)	$\alpha(\Delta t)^2$ (10^{-5} radians)	θ (10^{-5} radians)
0	7,700	65,410	410	0	64,990	3,120	408	3,600	46,790	1.90	1.19	0
0.0025	7,260	61,600	410	1,000	63,010	3,120	872	2,776	48,910	1.98	1.24	0.59
0.005	6,800	57,800	410	4,140	53,250	3,120	1,330	1,940	43,450	1.76	1.10	2.43
0.0075	6,400	54,400	410	9,120	44,870	3,120	1,795	993	39,860	1.62	1.01	5.37
0.010	5,960	50,600	410	15,850	33,340	3,120	2,340	8	33,300	1.35	0.84	9.32
0.0125	5,540	47,200	410	24,000	22,790	3,120	2,810	-882	27,250	1.11	0.69	14.11
0.015	5,100	43,300	410	33,200	9,690	3,120	3,220	-1,732	18,440	0.75	0.47	19.59
0.0175	4,720	40,000		41,800	-1,800					-0.065	-0.04	24.61
0.020	4,240	36,000		51,000	-15,000					-0.54	-0.34	30.12
0.0225	3,800	32,300		60,000	-27,700					-0.99	-0.62	35.29
0.025	3,675	31,200		67,800	-36,600					-1.48	-0.93	39.84
0.0275	2,970	25,300		73,800	-48,500					-1.74	-1.09	43.46
0.030	2,920	24,800		78,000	-53,200					-1.90	-1.18	45.99
0.0325	2,880	24,500		80,400	-55,900					-2.00	-1.25	47.34
0.035	2,860	24,300		80,500	-56,200					-2.01	-1.26	47.44*
0.0375	2,640	22,400										46.28

When sliding occurs, $\alpha = \dfrac{M_o - F_o\bar{y}}{I_o - m\bar{y}^2}$ $\quad M_o = F_f y_f - F_b y_b - F_p y_p - B\theta_o + mg\bar{y}\sin\theta$**

When no sliding occurs, $\alpha = M_o/I_o$ $\quad F_o = F_f - F_b - F_p - \mu mg - \mu V + \mu\dfrac{m_r\omega_o^{2\dagger}}{\bar{y}_r} + \dfrac{m_r\omega_o^2}{\bar{y}_r}\sin\theta^\dagger$

$I_o - m\bar{y}^2 = 27{,}957 - 129.56(5.06)^2 = 24{,}627\ \text{ft}^4$

* Maximum $\theta = 47.4(10)^{-5}$ radians.
** $(mg\bar{y}\sin\theta)$ - This item will not be used in the analysis because of relatively small magnitude of $\sin\theta$.
† These terms will not be used in the analysis because of small ω_o and θ values.

Table 9.16. Simultaneous Rigid Body Overturning and Sliding Analysis. Part II – Sliding

t (sec)	$\frac{F_o}{m}$	$\alpha_o \bar{y}$	$\ddot{x}_o$ (ft/sec^2)	$\ddot{x}_o(\Delta t)^2$ (10^{-5} ft)	x_o (10^{-5} ft)
0	27.8	9.6	18.2	10.4	0
0.0025	21.4	10.0	11.4	7.13	5.02
0.005	14.9	8.9	6.0	3.75	17.53
0.0075	7.67	8.20	-0.53	-0.33	33.76
0.010	0.06	6.83	-6.77	-4.23	49.66
0.0125	-6.8	5.00	-12.4	-7.75	61.33
0.015	-13.4	3.80	-17.20	-10.75	65.25*
0.0175					58.42

$$\ddot{x}_o = \frac{F_o}{m} - \alpha_o \bar{y} \qquad \text{(eq 9.5)}$$

* At time t = 0.0150 sec, x_o reaches its maximum value and $\dot{x}_o = 0$. Therefore sliding stops since $F_o/m > \alpha_o \bar{y}$ (see eq 9.7).

Table 9.17. Acceleration of Top of Shear Wall

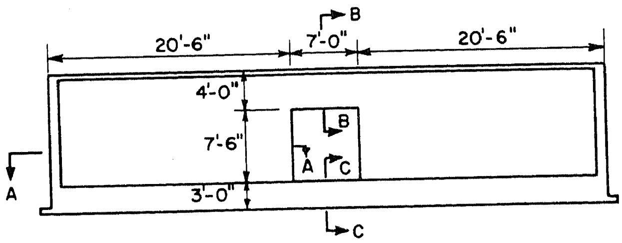
t (sec)	$\ddot{x}_o$ (ft/sec^2)	α (radian/sec^2)	13.96 α (ft/sec^2)	$\ddot{x}_r$ (ft/sec^2)
0	18.2	1.90	26.5	44.7
0.0025	11.4	1.98	27.6	39.0
0.005	6.0	1.76	24.6	30.6
0.0075	-0.53	1.62	22.6	22.1
0.010	-6.77	1.35	18.9	12.1
0.0125	-12.4	1.11	15.5	3.1
0.015	-17.20	0.75	10.5	-6.70
0.0175	0	-0.065	-0.91	-0.91
0.020	0	-0.54	-7.54	-7.54
0.0225	0	-0.99	-13.8	-13.8
0.025	0	-1.48	-16.0	-16.0
0.0275	0	-1.74	-24.3	-24.3
0.030	0	-1.90	-26.5	-26.5
0.0325	0	-2.00	-27.9	-27.9
0.035	0	-2.01	-28.1	-28.1

Acceleration ($\ddot{x}_r$) of top of shear wall = $\ddot{x}_o$ + 13.96 α (to be used in shear wall analysis in par. 9-14).

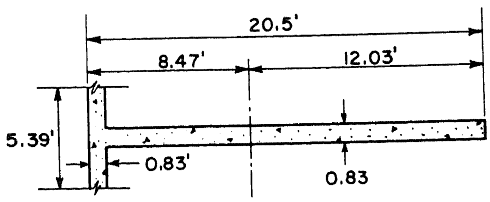
9-14 PRELIMINARY INVESTIGATION OF SHEAR WALLS. a. Determination of Resistance Functions. It is necessary to determine the resistance function, an expression relating the lateral deformation of the structure to the internal resistance which the structure develops to resist that deformation. This relationship is used for both the design of the shear walls and the determination of the magnitude of the overturning and sliding in the dynamic overturning and sliding analysis to be performed later in this example.

Since the transverse girders are relatively flexible compared to the shear walls, no rigid frame action develops; hence the only appreciable resistance to lateral deformation of the structure will be considered to be developed entirely within the shear walls.

Since the two end shear walls are slightly different from the interior shear walls, an investigation must be made of each type in order to determine the resistance of the entire structure. A moment and shear distribution of the shear walls will be performed in accordance with the procedures described in paragraphs 9-04b and 9-05 in order to obtain the resistance function of each wall. The resistances will then be combined in order to determine the resistance function of the entire structure acting as a whole. The preliminary shear wall investigation is based upon an assumed wall thickness of 10 in. which is considered the practical minimum thickness for reinforced concrete walls.



b. Interior Shear Wall. The resistance function for a typical interior shear wall as shown here is determined in the following paragraphs.



SECTION A-A

c. Section A-A (Shear Wall Section). The length of the front and back walls on each side of the shear walls which can be considered to act integrally with the shear wall as flanges should not be greater than (1) one-sixth height of wall,

since walls are designed for plastic action, (2) four times thickness of exterior wall, or (3) one-half the distance to adjacent shear wall, whichever is the least dimension.

$$b = 2\left(\frac{1}{6}\text{ height}\right) + t = 2\left(\frac{14.50 - 0.83}{6}\right) + 0.83 = 5.39 \text{ ft}$$

$$b = 8t + b' = 8(0.83) + 0.83 = 9(0.83) = 7.50 \text{ ft}$$

$5.39 < 7.50$ ft

Use 5.39 ft

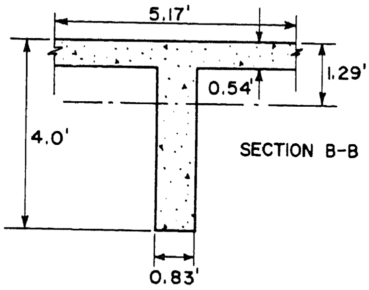
$$\bar{y} = \frac{20.5^2(0.83) + 0.83^2(4.56)}{2\left[0.83(2.05) + 0.83(4.56)\right]} = 8.47 \text{ ft}$$

$$I_{CG} = 20.5(0.83)\left[\left(\frac{20.5}{12}\right)^2 + 1.78^2\right] + 4.56(0.83)\left[\frac{0.83^2}{12} + 8.03^2\right]$$

$$= 646 + 244 = 890 \text{ ft}^4$$

$$\text{Area} = 20.5(0.83) = 17.0 \text{ ft}^2$$

d. Section B-B (Lintel Section).



SECTION B-B

$$b = 8t + b' = 8(0.54) + 0.83 = 5.17 \text{ ft}$$

$$\bar{y} = \frac{4.34(0.54)^2 + 0.83(4.0)^2}{2\left[0.83(4.0) + 0.54(4.34)\right]} = \frac{14.65}{2(5.66)} = 1.290$$

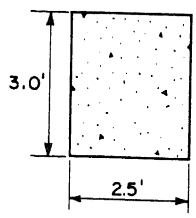
$$I_{CG} = 0.83(4.0)\left[\frac{4.0^2}{12} + 0.71^2\right] + 0.54(5.17)\left[\frac{0.54^2}{12} + 1.02^2\right] = 6.06 + 2.96 = 9.02 \text{ ft}^4$$

$$\text{Area} = 0.83(4.0) = 3.32 \text{ ft}^2$$

e. Section C-C (Footing Section).

$$\text{Area} = 2.5(3.0) = 7.5 \text{ ft}^2$$

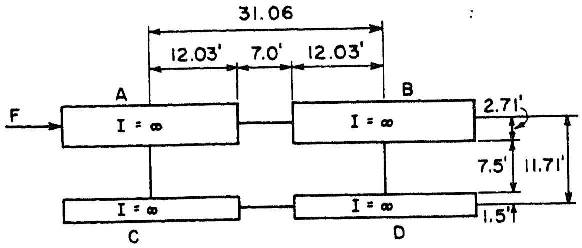
$$I_{CG} = \frac{7.5(3.0)^2}{12} = 5.62 \text{ ft}^4$$



SECTION C-C

f. Constants for Moment Distribution--Members CA and DB (Par. 9-04).

$L = 7.5 + 2.71 + 1.50 = 11.71$ ft



$x_1 = \frac{1.50}{11.71}, \; x_2 = \frac{9.0}{11.71}$

$x_1 = 0.128 \; , \; x_2 = 0.770$

$a_1 = x_2 - x_1 = 0.770 - 0.128$
$= 0.642$

$a_2 = \frac{1}{2}\left(x_2^2 - x_1^2\right) = 0.5\;(0.770^2 - 0.128^2) = 0.298$

$a_3 = \frac{1}{3}\left(x_2^3 - x_1^3\right) = 0.33\;(0.770^3 - 0.128^3) = 0.151$

$S = \frac{I_o E}{L^2 A_o G} = \frac{890(2.2)}{11.71^2(17.0)} = 0.834 \; ; \; E/G = 2.2$

$a_3' = a_1 S + a_3 = (0.642)(0.834) + 0.151 = 0.687$

$C_1 = \frac{a_3'}{a_2 - a_3'} \quad C_2$

$C_2 = \frac{a_2 - a_3'}{a_1 a_3' - (a_2)^2} = \frac{0.298 - 0.687}{0.642(0.687) - (0.298)^2} = -1.10$

$C_1 = \frac{0.687}{0.298 - 0.687}\;(-1.10) = 1.95$

$C_3 = \frac{a_1 - 2a_2 + a_3'}{a_2 - a'_3}\;(C_2) = \left[\frac{0.642 - 2(0.298) + 0.687}{0.298 - 0.687}\right]\;(-1.10) = 2.12$

$K_{CA} = \frac{C_1}{4}\left(\frac{I_o}{L}\right) = \frac{1.95(890)}{4(11.71)} = 37.0$

$K_{AC} = \frac{C_3}{4}\left(\frac{I_o}{L}\right) = \frac{2.12(890)}{4(11.71)} = 40.30$

$C.O._{CA} = \frac{C_2}{C_1} = -\frac{1.10}{1.95} = -0.566$

$C.O._{AC} = \frac{C_2}{C_3} = -\frac{1.10}{2.12} = -0.527$

For unit lateral deflection of top of wall.

ψ_{CA} = chord rotation of CA = $1/L_{CA} = \psi_{AC}$

$$\mathrm{FEM}_{CA} = -4E(1 + C.O._{CA})K_{CA}\psi_{CA}$$
$$= -(4)(3)(10)^3 \left[1 + (-0.566)\right] (144)(37.0)/11.7$$
$$= -2.37(10)^6 \text{ ft-kips}$$

$$\mathrm{FEM}_{AC} = -4E(1 + C.O._{AC})K_{AC}\psi_{AC}$$
$$= -(4)(3)(10)^3 \left[1 + (-0.527)\right] (144)(40.3)/11.7$$
$$= -2.82(10)^6 \text{ ft-kips}$$

g. <u>Constants for Moment Distribution--Member AB (Par. 9-04).</u>

$$L = 7.0 + 12.03 + 12.03 = 31.06$$

$$x_1 = \frac{12.03}{31.06} = 0.387$$

$$x_2 = \frac{19.03}{31.06} = 0.613$$

$$a_1 = x_2 - x_1 = 0.613 - 0.387 = 0.226$$

$$a_2 = \frac{1}{2}\left(x_2^2 - x_1^2\right) = 0.5\left[(0.613)^2 - (0.387)^2\right] = 0.1132$$

$$a_3 = \frac{1}{3}\left(x_2^3 - x_1^3\right) = \frac{1}{3}\left[(0.613)^3 - (0.387)^3\right] = 0.0576$$

$$S = \frac{I_o E}{L^2 A_o G} = \frac{9.02(2.2)}{(31.06)^2\ 3.32} = 0.00620\ ;\ E/G = 2.2$$

$$a_3' = a_1 S + a_3 = 0.2260(0.00620) + 0.0576 = 0.05901$$

$$C_2 = \frac{a_2 - a_3'}{a_1 a_3' - (a_2)^2} + \frac{0.1132 - 0.05901}{0.2260(0.05901) - (0.1132)^2} = 108.3$$

$$C_1 = \frac{a_3'}{a_2 - a_3'}\ C_2 = \frac{0.05901}{0.1132 - 0.05901}(108.3) = 118.0$$

For symmetrical members: $K_{AB} = K_{BA} = \frac{C_1}{4}\left(\frac{I_o}{L}\right)$

$$C.O._{AB} = C.O._{BA} = \frac{C_2}{C_1}$$

$$K_{AB} = \frac{118.0}{4}\left(\frac{9.02}{31.06}\right) = 7.86$$

$$C.O._{AB} = \frac{C_2}{C_1} = \frac{108.3}{118} = 0.919$$

h. <u>Constants for Moment Distribution--Member CD (Par. 9-04).</u>

a_1, a_2, and a_3, same as for member AB

$$S = \frac{I_o E}{L^2 A_o G} = \frac{(5.62)\ 2.2}{(31.06)^2\ 7.5} = 0.001710 \ ;\ E/G = 2.2$$

$$a_3' = a_1 S + a_3 = 0.2260(0.001710) + 0.0576 = 0.0580$$

$$C_2 = \frac{a_2 - a_3'}{a_1 a_3' - (a_2)^2} = \frac{0.1132 - 0.0580}{0.2260(0.0580) - (0.1132)^2} = 172.2$$

$$C_1 = \frac{a_3'}{a_2 - a_3'} C_2 = \frac{0.0580}{0.1132 - 0.0576} 172.2 = 200$$

$$K_{CD} = \frac{200(5.62)}{4(31.06)} = 9.03$$

$$C.O._{CD} = \frac{C_2}{C_1} = \frac{172.2}{200.0} = 0.862$$

For antisymmetrical loading, K factors can be modified so that distribution can be performed on one-half structure.

$$K' = K(1 + C.O.)$$

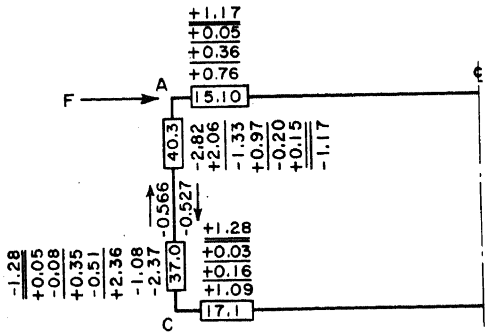
$$K'_{AB} = 7.86\ (1 + 0.919) = 15.10$$

$$K'_{CD} = 9.03\ (1 + 0.892) = 17.10$$

i. <u>Moment Distribution for Unit Lateral Displacement.</u>

$$F = 2(10)^6(1.28 + 1.17)/11.71 = 419{,}000 \text{ kips/ft}$$

The analysis performed assumed that the horizontal load on the shear wall was located at the centroid of member AB. The actual location, however, is at the top of the shear wall, hence a modification of the displacing force will be made to account for this action.



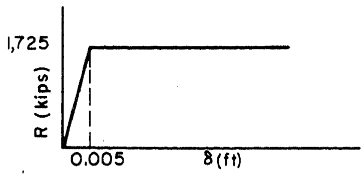
$$\delta = 1.0 + \frac{Fh}{AG} = 1.0 + \frac{(419{,}000)\ 4.0}{48.0\ (0.83)\ 1.36\ (10)^3\ 144} = 1.0 + 0.214 = 1.214 \text{ ft}$$

Elastic $k = \frac{F}{\delta} = \frac{419,000}{1.214} = 345,000$ kips/ft

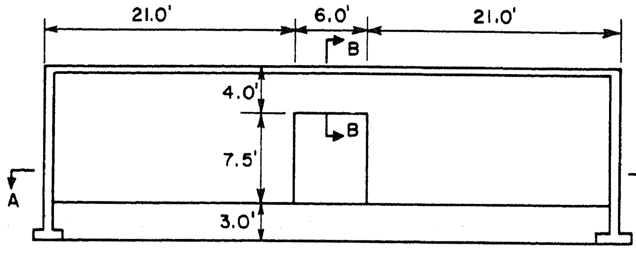
At first cracking: $R_c = 0.1\ f'_{dc}tL = 2(390)\ 0.83(18.5)\ \frac{144}{1,000} = 1,725$ kips (eq. 4.50)

$$\delta_e = \frac{R_c}{k} = \frac{1,725}{345,000} = 0.005 \text{ ft}$$

j. Resistance Function for One Interior Shear Wall. The load and deflection at first cracking define the resistance function in the elastic range as shown by the sketch on the left.



RESISTANCE FUNCTION–INTERIOR SHEAR WALL

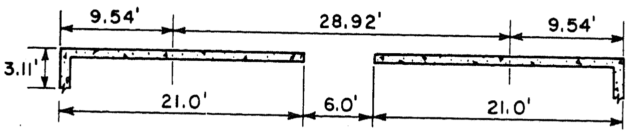


Elevation-End Shear Wall

k. End Shear Wall. The resistance function for the end shear wall shown in the sketch on the right is determined in the following paragraphs.

l. Section A-A (Shear Wall Section).

Effective flange is 1/6 height of wall $\therefore \frac{1}{6}(14.50 - 0.83) + 0.83 = 3.11$ ft



Section A-A Shear Wall Section

$$\bar{y} = \frac{\left[(21.0)^2\ 0.83 + (0.83)^2\ 2.28\right]}{2\left[(0.83)\ 21 + 0.83(2.28)\right]}$$

$$= 9.54 \text{ ft}$$

$$I_{cg} = 21(0.83)\left(\frac{21^2}{12} + 0.96^2\right) + 2.28(0.83)\left(\frac{0.83^2}{12} + 9.10^2\right)$$

$$= 657 + 157.0 = 814 \text{ ft}^4$$

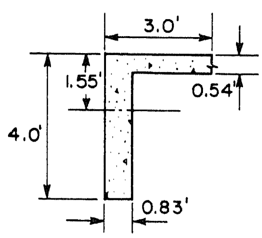
$A_w = (21.0)\ 0.83 = 17.42$ ft^2

m. Section B-B (Portal Section).

$b = 4(0.54) + 0.83 = 3.00$ ft

$A = 4.0(0.83) + 0.54(2.17) = 4.49$ ft^2

$\bar{y} = \frac{(3.32)\ 2.0 + (1.17)\ 0.27}{4.49} = 1.55$ ft



Sect. B-B Portal Section

$$I = 3.32\left(\frac{4.0^2}{12} + 0.45^2\right) + 1.17\left(\frac{0.54^2}{12} + 1.28^2\right)$$

$$= 5.07 + 1.94 = 7.01\ \text{ft}^4$$

n. <u>Footing Section (Same as for Interior Shear Wall).</u>

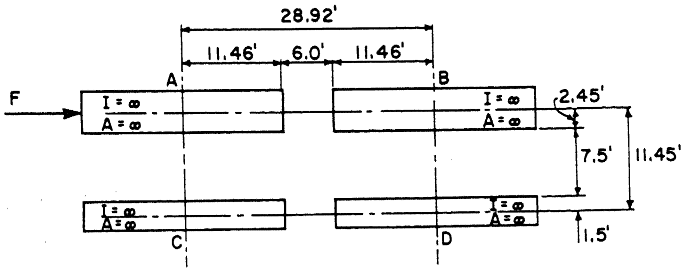
$I = 5.62\ \text{ft}^4$

$A = 7.5\ \text{ft}^2$

o. <u>Constants for Moment Distribution--Members CA and DB (Par. 9-04).</u>

$L = 7.5 + 2.45 + 1.50$

$= 11.45\ \text{ft}$



$$x_1 = \frac{1.50}{11.45} = 0.1310$$

$$x_2 = \frac{9.0}{11.45} = 0.785$$

$$a_1 = x_2 - x_1$$

$$= 0.785 - 0.131 = 0.654$$

$$a_2 = \frac{1}{2}\left(x_2^2 - x_1^2\right) = \frac{1}{2}\left[(0.785)^2 - (0.131)^2\right] = 0.2995$$

$$a_3 = \frac{1}{3}\left(x_2^3 - x_1^3\right) = \frac{1}{3}\left[(0.785)^3 - (0.131)^3\right] = 0.1606$$

$$S = \frac{I_oE}{L^2A_oG} = \frac{814\ (2.2)}{(11.45)^2(17.42)} = 0.784\ ;\ E/G = 2.2$$

$$a_3' = a_1S + a_3 = (0.6540)\ 0.784 + 0.1606 = 0.6726$$

$$C_2 = \frac{a_2 - a_3'}{a_1a_3' - (a_2)^2} = \frac{0.2995 - 0.6726}{(0.6540)\ 0.6726 - (0.2995)^2} = -1.022$$

$$C_1 = \frac{a_3'}{a_2 - a_3'}\ C_2 = \frac{0.6726}{0.2995 - 0.6726}\ (-1.022) = 1.86$$

$$C_3 = \frac{a_1 - 2a_2 + a_3'}{a_2 - a_3'}\ C_2 = \frac{0.6540 - 2(0.2995) + 0.6726}{0.2995 - 0.6726}\ (-1.022) = 2.06$$

$$K_{CA} = \frac{C_1}{4}\left(\frac{I_o}{L}\right) = \frac{1.86(814)}{4(11.45)} = 33.10$$

$$K_{AC} = \frac{C_3}{4}\left(\frac{I_o}{L}\right) = \frac{(2.06)\ 814}{4(11.45)} = 36.60$$

$$C.O._{CA} = \frac{C_2}{C_1} = \frac{-1.022}{1.86} = -0.5500$$

$$C.O._{AC} = \frac{C_2}{C_3} = \frac{-1.022}{2.06} = -0.5130$$

Unit lateral deflection of wall, ψ_{CA} = chord rotation of CA

$= 1/L_{CA} = \psi_{AC}$

$$FEM_{CA} = -4E(1 + C.O._{CA})K_{CA}\ \psi_{CA}$$
$$= -4(3)(10)^3 \left[1 + (-0.550)\right] 144(33.10)/11.45$$
$$= -2.25(10^6)\ \text{ft-kips}$$

$$FEM_{AC} = -4(3)(10)^3 \left[1 + (-0.5130)\right] 144(36.6)/11.45$$
$$= -2.68(10)^6\ \text{ft-kips}$$

p. <u>Constants for Moment Distribution--Member AB (Par. 9-04).</u>

$L = 11.46 + 6.0 + 11.46 = 28.92$

$$x_1 = \frac{11.46}{28.92} = 0.396$$

$$x_2 = \frac{17.46}{28.92} = 0.604$$

$$a_1 = x_2 - x_1 = 0.604 - 0.396 = 0.2080$$

$$a_2 = \frac{1}{2}\left[x_2^2 - x_1^2\right] = \frac{1}{2}\left[(0.604)^2 - (0.396)^2\right] = 0.05267$$

$$a_3 = \frac{1}{3}\left[x_2^3 - x_1^3\right] = \frac{1}{3}\left[(0.604)^3 - (0.396)^3\right] = 0.05267$$

$$S = \frac{I_oE}{L^2A_oG} = \frac{(7.01)\ 2.2}{(28.92)^2\ 3.32} = 0.00556\ ;\ E/G = 2.2$$

$$a_3' = a_1S + a_3 = (0.2080)\ 0.00556 + 0.05227 + 0.05342$$

$$C_2 = \frac{a_2 - a_3'}{a_1a_3' - (a_2)^2} = \frac{(0.1026 - 0.05342)}{(0.2080)\ 0.05342 - (0.1026)^2} = 89.9$$

$$C_1 = \frac{a_3'}{a_2 - a_3'}\ C_2 = \frac{0.05342}{0.1026 - 0.05342}\ (89.9) = 97.0$$

$$C.O._{AB} = \frac{C_2}{C_1} - \frac{89.9}{97.0} = 0.922$$

$$K_{AB} = \frac{C_1}{4}\left(\frac{I_o}{L}\right) = \frac{(97.0)\ 7.01}{4\ (28.92)} = 5.88$$

q. Constants for Moment Distribution--Member CD.

a_1, a_2, and a_3 same as for member AB

$$S = \frac{I_o E}{L^2 A_o G} = \frac{(5.62)\ 2.2}{(28.92)^2\ 7.5} = 0.001970 \ ; \ E/G = 2.2$$

$$a_3' = a_1 S + a_3 = 0.2080\ (0.00197) + 0.05267 = 0.05308$$

$$C_2 = \frac{a_2 - a_3'}{a_1 a_3' - (a_2)^2} = \frac{(0.1026 - 0.05308)}{(0.208)\ 0.05303 - (0.1026)^2} = 97.4$$

$$C_1 = \frac{a_3'}{a_2 - a_3'}\ C_2 = \frac{0.05308}{0.1026 - 0.05303}\ (97.4) = 104.0$$

$$C.O._{CD} = \frac{97.4}{104} = 0.932$$

For antisymmetrical loading, K's can be modified so that distribution can be performed on one-half of structure,

$K' = K(1 + C.O.)$

$K'_{AB} = 5.88(1 + 0.922) = 11.30$

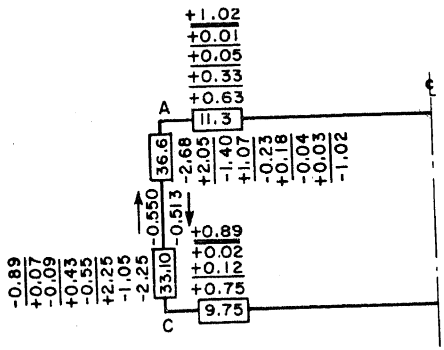
$K'_{CD} = 5.05(1 + 0.932) = 9.75$

r. Moment Distribution for Unit Lateral Displacement.

$F = 2(10)^6(1.02 + 0.89)/11.45$

$= 333{,}000$ ksf

In a manner identical with the interior shear wall, the displacing force will be modified to take account of the actual location of the horizontal force on shear wall.



$$\delta = 1.0 + \frac{Fh}{AG}$$

$$= 1.0 + \frac{(333{,}000)4.0}{48.0(0.83)\ 1.36(10)^3}\ 144 = 1.0 + 0.171 = 1.171 \text{ ft}$$

For end wall, k/2 shall be considered effective k because of blast load on end of building (see par. 9-07).

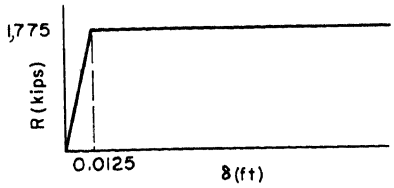
$$\text{Elastic } k = \frac{333{,}000}{1.171} = 284{,}000(0.5) = 142{,}000 \text{ kips/ft}$$

At first shear wall cracking $R_c = 0.1\ f'_{dc}\ tL$ (eq 4.50)

$$R_c = 2(390)\ 0.83(19.0)\ 144/1{,}000 = 1{,}775 \text{ kips}$$

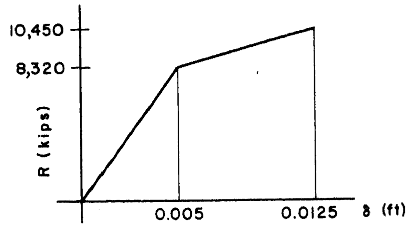
$$\delta_e = \frac{R_c}{k} = \frac{1{,}775}{142{,}000} = 0.0125$$

s. Resistance Function for One End Shear Wall. The load and deflection at first cracking define the resistance function in the elastic range as shown in the sketch on the left.



RESISTANCE FUNCTION-END SHEAR WALL

t. Resistance Function for Entire Building. The total resistance of the entire building is the total resistance of the four interior walls and the two end walls.



RESISTANCE FUNCTION-ENTIRE BUILDING

At $\delta = 0.005$ ft,

$$R = 4(1{,}725) + 2\left(\frac{0.0050}{0.0125}\right) 1{,}775 = 8{,}320 \text{ kips}$$

At $\delta = 0.0125$ ft, $R = 4(1{,}725) + 2(1{,}775) = 10{,}450$ kips

Equivalent dynamic system:

Total $k_1 = 2(k_{end}) + 4(k_{int})$

$k_1 = 2(142{,}000) + 4(345{,}000) = 1{,}664{,}000$ kips/ft

$m_2 = 36.82$ kip-sec^2/ft = mass of upper half of structure (par. 9-15a)

Elastic $T_n = 2\pi\sqrt{m_2/k_1} = 6.28\sqrt{36.82/1{,}664{,}000} = 0.0295$ sec

u. Dynamic Analysis by Numerical Integration.

Use acceleration impulse extrapolation method (par. 5-08d).

$$x_{n+1} = 2x_n - x_{n-1} + \ddot{x}_n(\Delta t)^2$$

$$\ddot{x}_n = \frac{P_n - R_n}{m_r} - \ddot{x}_r$$

($\ddot{x}_r$ is horizontal acceleration of top of shear wall as obtained from rigid body overturning and sliding analysis (see table 9.17).)

$P_n = 156.8\ V_2$ (front wall reaction at roof level). (Back wall react is zero until air blast acts on back wall.)

$R_n = k_1 x_n = 1{,}664{,}000\ x_n$ kips

using Δt = 0.0025 (approximately 1/10 T_n); $(\Delta t)^2/m = (0.0025)^2/36.82$
= 0.000000170

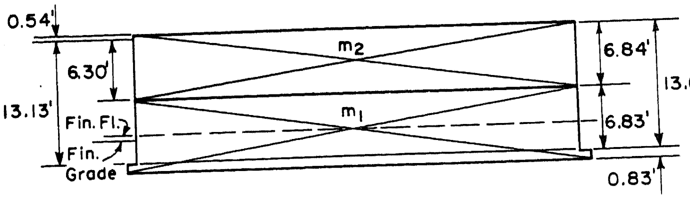
$$\ddot{x}_n(\Delta t)^2 = -0.00000625\ \ddot{x}_r + 0.000000170\ (P_n - R_n)$$

Table 9.18. Preliminary Shear Wall Analysis

t (sec)	V_2* (kips)	P** (kips)	R (kips)	P - R (kips)	$\ddot{x}_r$† (ft/sec²)	$\ddot{x}_{net}(\Delta t)^2$ (10^{-5} ft)	x (10^{-5} ft)
0	4.96	777	0	777	44.7	-14.8	0
0.0025	6.24	976	0	976	39.0	-7.8	0††
0.005	10.21	1,600	0	1,600	30.6	+8.1	0††
0.0075	15.92	2,500	135	2,365	22.1	+26.4	8.1
0.010	17.80	2,790	710	2,080	12.1	+27.8	42.6
0.0125	17.51	2,750	1,748	1,002	3.1	+15.1	104.9
0.015	17.20	2,695	3,040	-345	-6.70	-1.7	182.3
0.0175	16.90	2,650	4,300	-1,680	-0.91	-27.5	258.0
0.020	16.62	2,610	5,106	-2,490	-7.54	-37.7	306.2
0.0225	16.31	2,560	5,280‡	-2,720	-13.80	-37.6	316.7
0.025					-16.00		289.6

* Obtained from table 9.9.
** P = 156.8 V_2.
† $\ddot{x}_r$ is tabulated in table 9.17.
†† In the first two intervals, $\ddot{x}_{net}$ is negative because of the relatively large $\ddot{x}_r$. Since this cannot occur in a one-story structure, $\ddot{x}_{net}$ is set equal to 0 during these time intervals.
‡ Maximum shear wall resistance developed = 5,280 kips. This is less than the maximum shear wall strength, hence the wall sections are satisfactory.

9-15 DYNAMIC OVERTURNING AND SLIDING INVESTIGATION. In order to include the effect of lateral displacement of the upper half of the structure with respect to the lower portion during overturning and sliding it is necessary to perform a simultaneous dynamic overturning and sliding analysis (par. 9-06c).



This can be performed only after the preliminary shear wall investigation so that the shear wall stiffnesses can be used in determining the net force on the upper and lower portions of the structure

during the overturning and sliding action.

The structure must be represented by two masses, m_1 and m_2, as shown in the sketch on the preceding page.

a. Computation of Mass, Upper Portion of Structure. It is necessary to obtain the magnitude and locate the c.g. of each mass. m_2 will be obtained using a ratio of total masses contained in table 9.11.

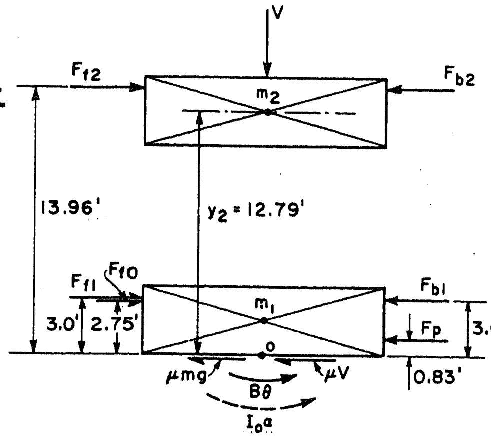


Table 9.19. Computation of m_2 and y_2

Element	m	Ratio	m_2	$\bar{y}$ from top	$m_2\bar{y}$
Front wall	7.96	$\frac{6.30}{13.13}$	3.81	3.69	14.05
Rear wall	7.96	$\frac{6.30}{13.13}$	3.81	3.69	14.05
End walls	4.32	$\frac{6.30}{13.13}$	2.06	3.69	7.50
Shear walls	8.40	$\frac{6.30}{13.13}$	4.02	3.69	14.85
Roof slab	18.85	1.0	18.85	0.27	5.09
Trans. girder	1.84	1.0	1.84	1.14	2.10
Long. girder	1.38	1.0	1.38	1.14	1.57
Columns	2.04	$\frac{6.30}{12.17}$	1.05	3.69	3.88
			Σ36.82		Σ63.09

$\bar{y}$ from top $= \frac{63.09}{36.82} = 1.71$ ft.

y_2 = height $-\ \bar{y}$ from top $= 14.50 - 1.71 = 12.79$ ft.

b. Computation of Mass, Lower Portion of Structure.

$m = 129.56$ kip-sec^2/ft

$m_2 = 36.82$

$m_1 = 92.74$

$$m_1 y_1 + m_2 y_2 = m\bar{y}$$

$$y_1 = \frac{m\bar{y} - m_2 y_2}{m_1} = \frac{656.81 - (36.82)(12.79)}{92.74}$$

$$y_1 = 2.00 \text{ ft}$$

c. Dynamic Analysis. The dynamic analysis will be performed by a concurrent numerical integration of equations (9.8), (9.9), and (9.10). When sliding does not occur, equation (9.11) will be used in place of equation (9.8).

$$\alpha_o = \frac{M_o - (F_1 - R_1)\, y_1 - (F_2 - R_2)\, y_2}{I_o - m_1 y_1^2 - m_2 y_2^2} \qquad \text{(eq 9.8)}$$

$$\ddot{x}_o = \frac{(F_1 - R_1)}{m_1} - \alpha_o y_1 \qquad \text{(eq 9.9)}$$

$$\ddot{x}_2 = \frac{(F_2 - R_2)}{m_2} - (\ddot{x}_o + \alpha_o y_2) \qquad \text{(eq 9.10)}$$

$$\alpha_o = \frac{M_o - (F_2 - R_2)\, y_2}{I_o - m_2 y_2^2} \qquad \text{(eq 9.11)}$$

d. Shear Wall Displacement. (See table 9.23.)

e. Soil Pressure Investigation. In order to determine maximum and minimum soil pressures at front and rear wall footings it will be necessary to summarize the dynamic reactions on these elements. Since a portion of the building has been considered to offer no resistance to overturning, it will be necessary to consider only those footings of the portion of the building which rotates.

Area of footings of rotating portion of building:

Front and rear wall footings	2(2.9)158.5 =	792.0 ft^2
Shear and end wall footings	6(2.5)44.75 =	672.0
Attached columns footings	20(2.67)1.67 =	89.4
	Total area	1,553.4 ft^2

f. Dead Load Soil Pressure. Weight of overturning portion of building is obtained from table 9.11, par. 9-13a. This is summarized on page 81.

Table 9.20. Tabulation of M_o (to be Used in Equations (9.8) and (9.11))

$$\Sigma M_o = (F_{f2} - F_{b2})\, y_{f2} + (F_{f1} - F_{b1})\, y_{f1} + F_{fo}\, y_{fo} - F_p y_p - \theta_o B$$

$$F_p = 492 \text{ kips (par. 9-13c)}$$

$$F_p = 0 \text{ if the structure does not slide}$$

t (sec)	V_{w2}* (kips)	$F_{f2} - F_{b2}$** (kips)	$(F_{f2} - F_{b2})y_{f2}$ (ft-kips)	V_{w1}† (kips)	$F_{f1} - F_{b1}$†† (kips)	$(F_{f1} - F_{b1})y_{f1}$ (ft-kips)	F_{fo}‡ (kips)	$(F_{fo})y_{fo}$ (ft-kips)	$F_p y_p$ (ft-kips)	θB‡‡ (ft-kips)	M_o (ft-kips)
0	4.96	778	10,890	4.96	778	2,330	322	888	0	0	14,108
0.0025	6.24	978	13,650	6.24	978	2,930	304	836	0	102	17,314
0.005	10.21	1,605	22,400	10.21	1,605	4,820	285	784	0	494	27,510
0.0075	15.92	2,500	34,900	15.92	2,500	7,500	268	736	0	1,480	41,656
0.010	17.80	2,790	38,800	17.80	2,790	8,360	251	720	0	3,540	44,340
0.0125	17.51	2,575	37,300	17.51	2,575	7,720	232	638	0	7,160	38,498
0.015	17.20	2,700	37,700	17.20	2,700	8,090	214	590	0	12,780	33,600
0.0175	16.90	2,650	37,000	16.90	2,650	7,950	196	540	0	20,420	25,070
0.020	16.62	2,610	36,400	16.62	2,610	7,820	177	486	0	29,900	14,806
0.0225	16.31	2,560	35,800	16.31	2,560	7,670	160	440	0	40,600	3,310
0.025	16.01	2,510	35,000	16.01	2,510	7,720	154	424	0	51,300	-8,156
0.0275	15.72	2,460	34,300	15.72	2,460	7,380	124	341	0	61,400	-19,379
0.030	15.66	2,450	34,200	15.66	2,450	7,350	122	333	0	69,700	-27,817
0.0325	14.36	2,250	31,400	14.36	2,250	6,750	121	332	0	76,400	-37,918
0.035	11.78	1,848	25,800	11.78	1,848	5,540	120	330	0	81,000	-49,330
0.0375	8.47	1,328	18,500	8.47	1,328	3,980	110	302	0	83,700	-60,918
0.040	4.91	770	10,750	4.91	770	2,210	104	286	0	85,000	-71,754
0.0425	1.93	302	4,220	1.93	302	906	101	278	0	84,600	-79,197
0.045	-0.10	-15.7	-220	-0.10	-15.7	-47.1	97	266	0		

* V_{w2} = algebraic sum of values of top reaction per foot of wall panels on front and rear walls (tables 9.9 and 9.10).

** $F_{f2} - F_{b2} = 156.8\ V_{w2}$ = net horizontal blast force on structure at top of wall.

† V_{w1} = same as V_{w2} for this example since wall slabs are considered simply supported.

†† $F_{f1} - F_{b1}$ = same as $F_{f2} - F_{b2}$ for this example.

‡ F_{fo} = blast load on wall of structure at intersection of floor and wall = $(156.8 \text{ ft})(0.5 \text{ ft}) \frac{144}{1{,}000} (\bar{P}_{front} - \bar{P}_{back})$. $(\bar{P}_{front} - \bar{P}_{back})$ is obtained from figure 9.20.

‡‡ $B = 17(10)^7$ ft-kips/radian (par. 9-13). θ is obtained from table 9.22.

Table 9.21. Tabulation of Vertical Blast Loads

t (sec)	V_A* (kips)	$52V_A$ (kips)	V_B* (kips)	$52V_B$ (kips)	$\overline{P}_{roof}$** (kips)	V_C† (kips)	V†† (kips)	μV (kips)
0	0	0	0	0	0	0	0	0
0.0025	1.17	60.9	2.40	125	0.60	184	270	202
0.005	2.92	154.0	6.27	326	1.20	368	848	636
0.0075	5.69	296.0	12.68	660	1.80	552	1,508	1,130
0.010	9.42	490	21.58	1,120	2.50	770	2,380	1,785
0.0125	13.84	722	32.05	1,670	3.10	950	3,342	2,510
0.015	18.02	940	42.55	2,210	3.65	1,120	4,270	3,200
0.0175	21.68	1,130	51.80	2,700	4.30	1,320	5,150	3,860
0.020	24.75	1,285	58.40	3,040	5.00	1,535	5,850	4,380
0.0225	26.45	1,372	62.05	3,220	5.60	1,720	6,312	4,740
0.025	27.70	1,440	64.80	3,370	6.15	1,890	6,700	5,030
0.0275	28.90	1,500	67.60	3,520	6.75	2,070	7,090	5,320
0.030	30.90	1,605	71.60	3,720	7.45	2,290	7,615	5,720
0.0325	33.80	1,755	78.30	4,070	8.05	2,480	8,305	6,240
0.035	37.30	1,935	86.70	4,500	8.65	2,660	9,095	6,820
0.0375	39.20	2,040	91.30	4,740	8.62	2,650	9,430	7,060
0.040	39.00	2,030	93.20	4,850	8.45	2,600	9,480	7,100
0.0425	40.60	2,110	94.80	4,920	8.35	2,560	9,590	7,200
0.045	41.02	2,130	95.70	4,960	8.30	2,550	9,740	7,300
0.0475	40.92	2,120	95.40	4,950	8.20	2,520	9,590	7,180
0.050					8.09	2,480		

* V_A and V_B obtained from table 9.3.

** $\overline{P}_{roof}$ obtained from figure 9.18.

† V_C = roof pressure on corridor slab and portions of roof directly above girders and walls

$$= \left[(7.0)(156.8) + 4(0.83)(156.8) + (14)\ 2(18.33)1.0\right] \frac{144}{1,000} \overline{P}_{roof}$$

$$= 307\ \overline{P}_{roof}\ .$$

†† $V = 52V_A + 52V_B + V_C$.

Table 9.22. Simultaneous Dynamic Overturning and Sliding Analysis. Part I – Overturning

$$F_2 - R_2 = (F_{f2} - F_{b2}) - R$$

$$F_1 - R_1 = (F_{f2} - F_{b2}) + F_{fo} - F_p - \mu V - \mu mg + R$$

t (sec)	R* (kips)	μV (kips)	μmg (kips)	$F_1 - R_1$ (kips)	$(F_1 - R_1)y_1$ (ft-kips)	$F_2 - R_2$ (kips)	$(F_2 - R_2)y_2$ (ft-kips)	α_o** (radians/sec²)	$\alpha_o(\Delta t)^2$ (10^{-5} radians)	θ (10^{-5} radians)
0	0	0	3,120	-2,512	-5,024	778	9,960	0.19/2	0.06	0
0.0025	97	202	3,120	-1,945	-3,890	881	11,280	0.27	0.17	0.06
0.005	406	636	3,120	-1,451	-2,902	1,199	15,340	0.56	0.35	0.29
0.0075	980	1,130	3,120	-1,803	-3,606	1,520	19,460	1.01	0.63	0.87
0.010	1,852	1,740	3,120	-268	-526	938	12,000	1.47	0.92	2.08
0.0125	2,790	2,510	3,120	-183	-366	-215	-2,750	1.87	1.17	4.21
0.015	3,420	3,200	3,120	-226	-452	-720	-9,200	1.95	1.22	7.51
0.0175	3,580	3,860	3,120	-814	-1,628	-930	-11,900	1.68	1.05	12.03
0.020	3,270	4,380	3,120	-1,673	-3,346	-660	-8,440	1.06	0.66	17.60
0.0225	2,620	4,740	3,120	-2,020	-4,040	-60	-766	0.19	0.12	23.83
0.025	1,927	5,030	3,120	-3,691	-7,382	+583	+7,460	-0.71	-0.46	30.18
0.0275	1,500	5,320	3,120	-4,446	-8,892	960	12,280	-1.44	-0.90	36.07
0.030	1,535	5,720	3,120	-4,867	-9,734	915	11,700	-1.81	-1.14	41.06
0.0325	2,065	6,240	3,120	-5.139	-10,278	+185	2,360	-1.84	-1.15	44.91
0.035	2,890	6,820	3,120	-5,392	-10,784	-1,042	-13,300	-1.65	-1.03	47.61
0.0375	3,650	7,060	3,120	-6,162	-12,324	-2,322	-29,700	-1.43	-0.89	49.28
0.040	3,920	7,100	3,120			-3,150	-40,200	-1.44	-0.90	50.06
0.0425	3,510	7,200	3,120			-3,208	-41,000	-1.74	-1.09	49.94
0.045		7,300	3,120							48.85
0.0475		7,180	3,120							

* R = 1,664,000 x kips (par. 9-14u).

$I_o - m\bar{y}_1{}^2 - m_2\bar{y}_2{}^2 = 27{,}957 - (92.74)(2.0)^2 - (36.82)(12.79)^2 = 18{,}227$

$I_o - m_2\bar{y}_2 = 27{,}957 - (36.82)(12.79)^2 = 21{,}937$

** These values assume no sliding and are obtained using equation (9.11).

Table 9.23. Simultaneous Dynamic Overturning and Sliding Analysis. Part II – Sliding

t (sec)	$(F_1 - R_1)/m_1$ (ft/sec²)	$\alpha_o y_1$ (ft/sec²)	$\ddot{x}_o$ (ft/sec²)	x_o (ft)	$(F_2 - R_2)/m_2$ (ft/sec²)	$\alpha_o y_2$ (ft/sec²)	$\ddot{x}_2$ (ft/sec²)	$\ddot{x}_2(\Delta t)^2$ (10^{-5} ft)	x_2 (10^{-5} ft)
0	-27.1	+0.96	-28.06	0	21.1	2.4	18.7	11.7/2	0
0.0025	-21.0	+1.00	-22.00	0	24.0	3.5	20.5	12.8	5.8
0.005	-15.7	+1.66	-17.36	0	32.6	7.2	25.4	15.9	24.4
0.0075	-19.5	+2.86	-22.36	0	41.3	12.9	28.4	17.8	58.9
0.010	-5.67	+3.50	-9.17	0	25.4	18.8	6.6	4.1	111.2
0.0125	+3.95	+4.46	-0.54	0	-5.8	23.9	-29.7	-18.5	167.6
0.015	-4.88	+4.62	-9.50	0	-19.6	25.0	-44.6	-27.8	205.4
0.0175	-17.58	+4.40	-21.97	0	-25.2	21.5	-46.7	-29.2	215.4
0.020	-36.20	+3.44	-39.64	0	-17.9	13.6	-31.5	-19.7	196.2
0.0225	-43.60	+1.84	-45.44	0	-1.6	2.4	-4.0	-2.5	157.3
0.025	-79.70	+0.34	-79.36	0	+15.9	-9.1	+25.0	+15.6	115.9
0.0275	-95.80	-0.86	-94.94	0	26.2	-18.4	44.6	27.8	90.1
0.030	-105.0	-1.80	-103.20	0	24.8	-23.2	48.0	30.0	92.1
0.0325	-111.0	-0.74	-110.26	0	5.0	-23.5	28.5	17.8	124.1
0.035	-116.2	-1.50	-114.70	0	-28.3	-21.1	-7.2	-4.5	173.9
0.0375	-133.0	-1.06	-131.94	0	-64.1	-18.3	-45.8	-28.6	219.2
0.040				0	-85.5	-18.4	-67.1	-41.9	235.9*
0.0425				0					210.7
0.045									
0.0475									

* Maximum shear wall displacement.

	Mass kip-sec^2/ft
Front wall	7.96
Rear wall	7.96
End walls	4.32
Shear walls	8.40
Roof slab	12.09
Transverse girders	0.92
Longitudinal girders	0.85
Columns	1.13
Exterior wall footing	3.90
Attached footing	0.34
Shear and exterior wall footing	1.75
Total mass	39.01

$$W = mg = (39.01)\ 32.2 = 1{,}255 \text{ kips}$$

To the above tabulation will be added the weight of soil and slab directly above that portion of the footings which participate in the rotation.

	Dimensions		Volume (cu ft)		Weight of earth (kips)
Earth above shear wall footings	4(1.67)1.67(46.0)	=	512	=	51.2
Earth above end wall footings	2(0.83)1.67(46.0)	=	127	=	12.7
Earth above front and rear wall footings	2(0.83)1.67(156)	=	432	=	43.2
Total weight of earth					107.1
Slab above shear wall footings	4(1.67)0.5(46.0)	=	154	=	23.1
Slab above end wall footings	2(0.83)0.5(46.0)	=	38	=	5.7
Slab above front and rear wall footings	2(0.83)0.5(156)	=	129	=	19.4
Total weight of slab					48.2

$$1{,}255 + 107 + 48 = 1{,}410 \text{ kips}$$

$$\text{Average dead load soil pressure} = \frac{1{,}410}{1{,}553.4} = 0.909 \text{ ksf}$$

g. Normal Blast Load Soil Pressure. The blast load pressures will consist of both a pressure due to normal blast load and due to rotation of

Table 9.24. Summary of Earth Pressures

The maximum soil pressure consists of a summation of pressure due to loads V plus dead load plus moment ΘB.

The minimum soil pressure consists of a summation of pressure due to loads V plus dead load minus moment ΘB.

t (sec)	V'_a* (kips)	V'_b** (kips)	V'_c† (kips)	V†† (kips)	Bθ‡ (ft-kips)	Dead Load Pressure (ksf)	Blast Load Pressure (ksf)‡‡	Overturning Pressure (ksf)	Minimum Pressure (ksf)	Maximum Pressure (ksf)
0	0	0	0	0	0	0.909	0	0	0.91	0.91
0.0025	42.2	86.5	109	237.7	102	0.909	0.15	0.00	1.06	1.06
0.005	105.0	226.0	218	549.0	494	0.909	0.35	0.02	1.24	1.28
0.0075	205	457	328	990.0	1,480	0.909	0.64	0.06	1.49	1.61
0.010	338	776	456	1,570	3,540	0.909	1.01	0.15	1.77	2.07
0.0125	407	1,155	565	2,217	7,160	0.909	1.43	0.31	2.03	2.65
0.015	650	1,535	665	2,850	12,780	0.909	1.84	0.55	2.20	3.30
0.0175	782	1,870	785	3,437	20,420	0.909	2.21	0.88	2.24	4.00
0.020	890	2,100	910	3,860	29,900	0.909	2.48	1.29	2.10	4.68
0.0225	955	2,240	1,020	4,215	40,600	0.909	2.71	1.75	1.87	5.37
0.025	998	2,330	1,120	4,448	51,300	0.909	2.86	2.22	1.55	5.99
0.0275	1,040	2,440	1,230	4,710	61,400	0.909	3.03	2.64	1.30	6.58
0.030	1,112	2,580	1,360	5,052	69,700	0.909	3.26	2.99	1.18	7.16
0.0325	1,220	2,820	1,465	5,505	76,400	0.909	3.55	3.28	1.18	7.74
0.035	1,345	3,120	1,575	6,040	81,000	0.909	3.89	3.48	1.32	8.28
0.0375	1,410	3,290	1,570	6,270	83,700	0.909	4.04	3.59	1.36	8.54
0.040	1,405	3,360	1,540	6,305	85,000	0.909	4.07	3.65	1.33	8.63
0.0425	1,460	3,420	1,520	6,400	84,600	0.909	4.13	3.64	1.40	8.68
0.045	1,480	3,450	1,510	6,440	83,000	0.909	4.14	3.57	1.48	8.62
0.0475	1,472	3,440	1,495	6,407		0.909	4.13			

* $V'_a = 36\ V_a$ (table 9.3).

** $V'_b = 36\ V_b$ (table 9.3).

† $V'_c = 182\ \overline{P}_{roof}$ (fig. 9-18).

†† $V = V'_a + V'_b + V'_c$ total vertical blast load.

‡ Θ obtained from table 9.22.

‡‡ Blast load pressure = V/1553.4 ksf.

the structure. It is necessary to determine the blast load on that portion of the building which is considered to rotate. This will be obtained by subtracting the blast load on the portion of the structure not rotating from the total blast load, table 9.21.

Portion of Total Vertical Blast Loading Not Acting upon Rotating Part of Structure	Net Vertical Blast Load upon Rotating Part of Structure
16 "a" edge slab reactions	$52 - 16 = 36\ V_A$
16 "b" edge slab reactions	$52 - 16 = 36\ V_B$
8 bays of corridor slab plus blast load directly on girders $= \dfrac{872(144)\bar{P}_{roof}}{1,000} = 125\ \bar{P}_{roof}$	$307 - 125 = 182\ \bar{P}_{roof}$

These will be tabulated in table 9.24.

h. Overturning Blast Load Soil Pressure.

$$\text{Soil pressure due to overturning} = \frac{\theta Bc}{I}$$

c = distance from axis of rotation to center of front or rear wall footing = 23.7 ft

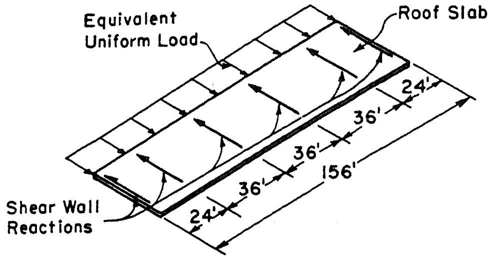
$$I = 6\left(\frac{1}{12}\right) 2.5(44.75)^3 + 2\left[\frac{158.5(2.5)^3}{12} + (396)(23.6)^2\right] = 552.165 \text{ ft}^4$$

$$\text{Overturning pressure} = \frac{23.7\ \theta B}{552.165} = 4.30(10)^{-5}\ \theta B \text{ kips/ft}^2$$

$B = 17.0(10)^7$ ft-kips (see par. 9-13b).

i. Summary. The maximum dynamic shear wall displacement computed in table 9.23 (x_2 = 0.00236 ft) is less than the deflection at first cracking in the interior shear walls (δ_e = 0.005 ft, see par. 9-14i) and in the end shear walls (δ_e = 0.0125 ft, see par. 9-14r). The 10-in.-minimum shear wall thicknesses are therefore satisfactory. The maximum earth pressure under the front and rear walls as determined from the tabulation in table 9.24 is 8.68 kips/sq ft. This pressure is well below the ultimate load-bearing capacity of 30 kips/sq ft, hence the footing design is satisfactory.

9-16 ROOF AND FLOOR SLAB DESIGN (Deep Beam Action). a. Roof Slab Analysis. Using the resistance value R obtained from the maximum shear



wall displacement in table 9.22 of the dynamic overturning and sliding investigation (par. 9-15c) as loads applied to the roof slab (acting as a deep beam) it is possible to obtain an equivalent load acting upon the roof slab.

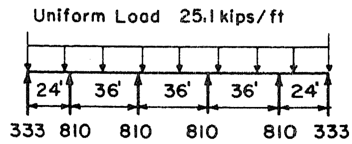
b. Loads on Roof Slab. Maximum R = 3,920 kips (t = 0.040 sec); uniform load = 3,920/156 = 25.1 kips/ft. The distribution of this uniform load to the roof slab will be considered in two different ways:

(i) Proportioning in accordance with relative resistance offered by end and interior shear walls (par. 9-14t).

(ii) Deep beam analysis by moment distribution taking into account shear deformation (par. 9-04b).

Whichever results in a more critical value will be used in subsequent shear wall analyses.

Proportioning shear wall loads in accordance with relative resistance, each end wall offers 142/1,664 of the total resistance (par. 9-14t) and each interior shear wall offers 345/1,664 of the total resistance.



End wall: 142/1,664 (3,920) = 333 kips

Interior wall: 345/1,664 (3,920) = 810 kips.

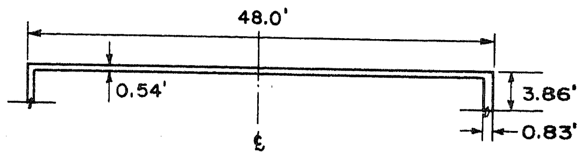
Shear wall loads obtained by relative shear wall stiffnesses are resisted by a uniform load of 25.1 kips/ft.

c. Properties of Roof Section (Deep Beam).

Effective flange width, b = 4t + b' = 4(0.83) + 0.54 = 3.86 ft

$$I_o = 2\left[3.32(0.83)^3/12 + (0.83)3.32(23.58)^2\right] + 0.54(48.0)^3/12$$
$$= 6{,}465 \text{ ft}^4$$

$A_o = 0.54(48.0) = 25.8 \text{ ft}^2$



Same for all members because of 4t + b' being controlling factor in flange width.

d. Constants for Moment Distribution--Center Spans (Par. 9-04).

$a_1 = 1$, $a_2 = 0.5$, $a_3 = 0.333$

$$S = \frac{I_o E}{L^2 A_o G} = \frac{(6,465)\ 2.2}{(36.0)^2\ 25.8} = 0.425\ ;\ E/G = 2.2$$

$$a_3' = a_1 S + a_3 = 0.425 + 0.333 = 0.758$$

$$C_2 = \frac{a_2 - a_3'}{a_1 a_3' - (a_2)^2} = \frac{0.5 - 0.758}{0.758 - (0.5)^2} = -0.508$$

$$C_1 = \left(\frac{a_3'}{a_2 - a_3'}\right) C_2 = \left(\frac{0.758}{0.5 - 0.758}\right)(-0.508) = +1.492$$

$$K = \frac{C_1}{4}\frac{I_a}{L} = \frac{1.492}{4}\left(\frac{6,465}{36.0}\right) = 67.0$$

$$C.O. = \frac{C_2}{C_1} = \frac{-0.508}{1.492} = -0.340$$

Modified K (symmetrical load) = K(1 - C.O.) = 67.0(1 + 0.340) = 89.7

e. <u>Constants for Moment Distribution--End Spans (Par. 9-04).</u>

$a_1 = 1,\ a_2 = 0.5,\ a_3 = 0.333$

$$S = \frac{(6,465)\ 2.2}{(24.0)^2\ 25.8} = 0.959$$

$$a_3' = 0.959 + 0.333 = 1.292$$

$$C_2 = \frac{0.5 - 1.292}{1.292 - (0.5)^2} = -0.760$$

$$C_1 = \left(\frac{1.292}{0.5 - 1.292}\right)(-0.760) = +1.240$$

$$K = \frac{1.240}{4}\left(\frac{6,465}{24.0}\right) = 83.6$$

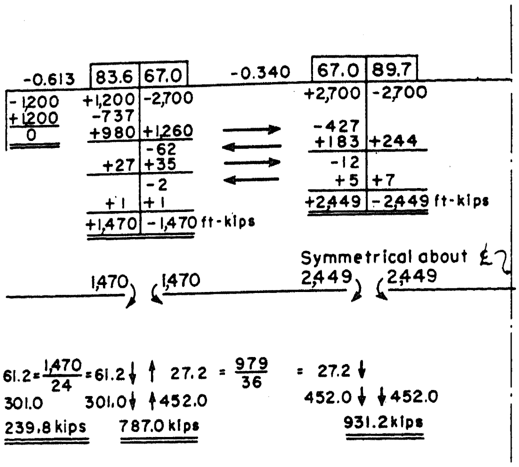
$$C.O. = \frac{-0.760}{1.240} = -0.613$$

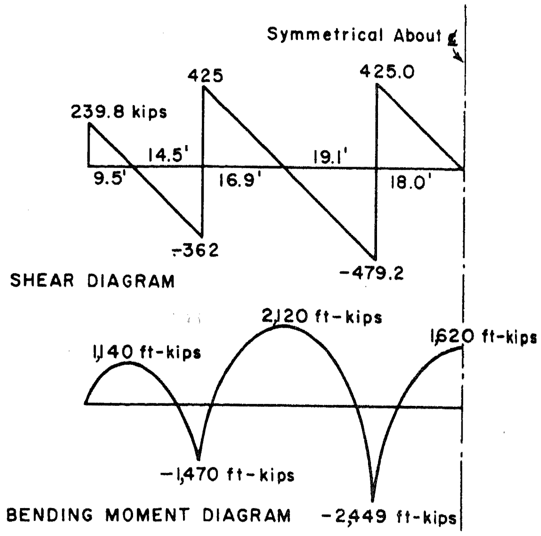
Modified K (end span) $= K\left[1 - (C.O.)^2\right] = 83.6\left[1 - (-0.613)^2\right] = 52.1$
(pinned support at end of span)

f. <u>Analysis of Deep Beam Using Moment Distribution (Par. 9-04).</u>

$FEM_{end} = 1/12\ wl^2 = 0.0833(25.1)(24.0)^2 = 1,200$ ft-kips

$FEM_{center} = 0.0833(25.1)(36.0)^2 = 2,700$ ft-kips





g. Shear Strength.

Shear strength (roof):

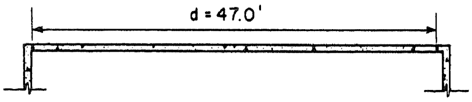
$$v = \frac{\text{max shear}}{\text{roof area}} = \frac{479.2(10)^3}{(25.8)\ 144} = 129 \text{ psi} < 0.10\ f'_{dc} = 390 \text{ psi}, \therefore \text{ OK}$$

Shear strength (wall):

$$v = \frac{\text{max reaction}}{\text{wall area}} = \frac{931.2}{0.83(48.0)\ 144} = 161 \text{ psi} < 0.10\ f'_{dc}$$

$$= 390 \text{ psi}, \therefore \text{ OK}$$

h. Steel Required for Bending. Maximum moment = 2,449.0 ft-kips (from bending moment diagram in par. 9-16f above). Assume this to be resisted entirely by steel placed within roof at front and back edges.



$$A_s = \frac{M}{f_{dy}d} = \frac{2,449}{(52.0)\ 47.0} = 1.00 \text{ in.}^2$$

Use three #6 bars

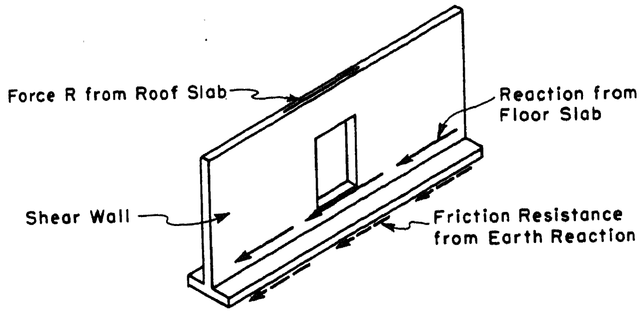
$A_s = 1.32 \text{ in.}^2$

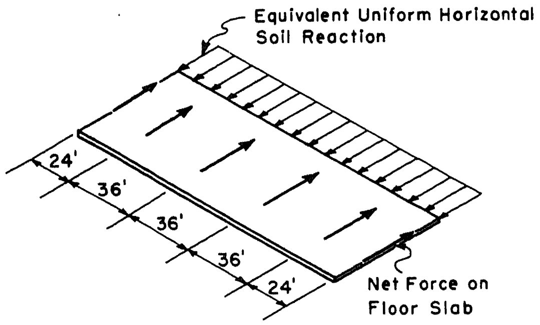
$\Sigma o = 7.1 \text{ in.}$

$$u = \frac{V}{\Sigma o jd} = \frac{479.2(1,000)}{7.1(0.875)\ 47(12)} = 137 \text{ psi} < 450 \text{ psi} = 0.15\ f'_c, \therefore \text{ OK}$$

i. Floor Slab Analysis. Since the reaction from the shear wall

deformation cannot be resisted completely by frictional force under the shear wall and earth pressures immediately behind the shear wall, it will be necessary to transmit the net force into the floor slab.





j. Loads on Floor Slab. In order to determine the magnitude of shears and moments in the floor slab an equivalent uniform load will be considered to act along the rear of the slab.

The net force acting on the floor slab is obtained by subtracting from the shear wall reactions the frictional force at time t = 0.040 sec (the maximum shear wall reactions occur at this time).

Frictional force on base of shear wall at time t = 0.040 sec (see table 9.24) = 0.75(2.5) 48.0(0.909 + 4.07) = 448 kips.

Net shear wall reactions upon floor slabs:

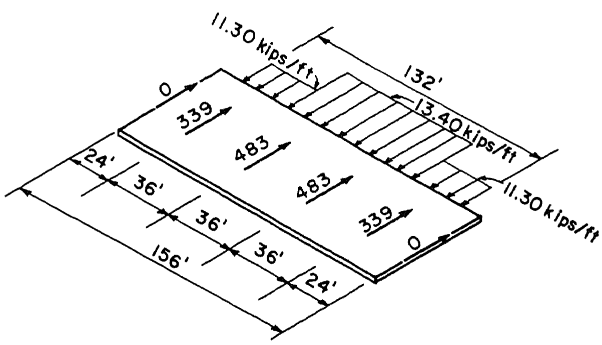
End shear walls	239.8 - 448 kips* = 0
First interior shear walls	787.0 - 448 kips = 339.0 kips
Center shear walls	931.2 - 448 kips = 483.0 kips

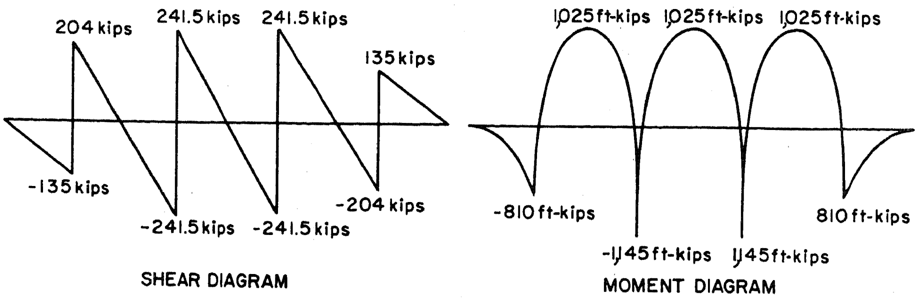
* It will be assumed that the net force applied by the end shear wall to the floor system will be zero because the earth reaction on this end wall will not exceed the shear wall reaction.

Assuming a distribution of uniform load for each half span on either side of the shear wall equal to the net force on the shear wall results in the following loading:

$$\frac{339}{\left(\frac{24 + 36}{2}\right)} = 11.30 \text{ kips/ft}$$

$$\frac{483}{\left(\frac{36 + 36}{2}\right)} = 13.40 \text{ kips/ft}$$





k. Steel Required for Bending.

$$A_s = \frac{M}{f_{dy}d} = \frac{1,145}{(52)\ 47.0} = 0.47 \text{ sq in.}$$

Use one #7 bar, $A_s = 0.60$ sq in.

$\Sigma o = 2.7$ in.

l. Shear Strength and Bond Stress.

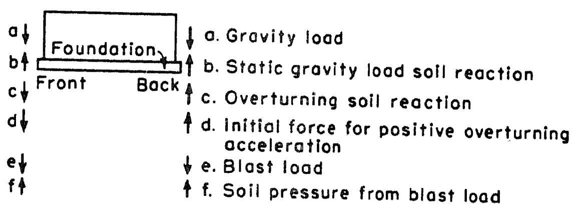
$$\text{Bond check } u = \frac{V}{\Sigma o j d} = \frac{241.5(1,000)}{2.7\left(\frac{7}{8}\right)\ 47(12)} = 181 \text{ psi} < 450 = 0.15\ f'_c \therefore \text{OK}$$

Dowels for shear. At the intersection of shear wall and floor slab, a maximum shear of 241.5 kips must be resisted. Since it is desirable to maintain integral construction of shear wall from top of footing to roof slab, it is necessary to design dowels to transmit this force across the construction joint.

Assuming the ratio of static yield stress in shear to yield stress in tension of $\frac{21}{41.6}$ (par. 4-03d), the steel necessary per foot of slab equals $\dfrac{241.5}{\left(\frac{21}{41.6}\right)(52)(47)} = 0.20 \text{ in.}^2/\text{ft}$

Use #4 at 12 in. for dowels. ($A_s = 0.20$ sq in./ft)

9-17 WALL ANALYSIS (Deep Beam Action). As the result of the overturning action of the structure there will be a shear force at the intersection of exterior wall and shear wall. In order to determine the critical location of this force (front or back wall) and its magnitude an analysis will be made below. The shear forces



and their direction acting on the shear wall are shown in the sketch on the preceding page.

These values will be computed and tabulated in order to determine which wall will be critical (front or back). It is necessary to first compute the mass and centroid of wall section being considered so the inertial force may be determined.

a. Computation of Mass and Centroid Location of 24-ft Section of Exterior Wall.

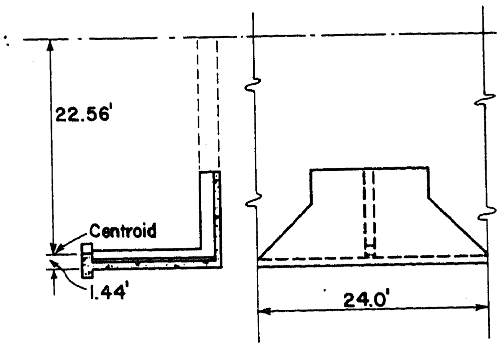


Table 9.25. Location of Centroid of Exterior Wall Section

	Volume ft^3	m	$\bar{y}$	$m\bar{y}$
Slab $0.54\left(\frac{19.17}{2}\right)12.0$	62.0	0.298	$\frac{19.17}{4}$	1.430
Column 1.0 (1.0) 12.17	12.17	0.057	2.0	0.114
Wall 0.83 (13.69) 23.17	263.50	1.230	0.41	0.504
Concrete footing 2.5 (0.83) 23.17	48.00	0.224	1.25	0.280
Column footing 2.67 (1.67) 0.83	3.70	0.017	2.09	0.036
Girder $1.33\left(\frac{19.17}{2}\right)1.5$	19.15	0.089	$\frac{19.17}{4}$	0.426
Slab 12.0 (3.0) 0.54	19.40	0.091	2.0	0.182
	427.92	2.006		2.972

$$\bar{y} = \frac{2.972}{2.006} = 1.44 \text{ ft}$$

Weight of 24-ft wall section = (427.92)(0.150) = 64.0 kips

Table 9.26. Tabulation of Forces on Shear Wall from 24-ft Exterior Wall Section

a. Gravity load, 64.0 kips

b. Gravity load soil reaction (table 9.24) = 0.909 $\left[(24.0)\ 2.5 + (2.67)\ 1.67\right]$ = 58.60 kips

c. Overturning soil reaction multiplied by footing area. Pressures from table 9.24. Area = (24.0) 2.5 + (2.67) 1.67 = 64.45 ft^2

d. Inertial force = $m r \alpha$ = 2.006 (24.0 - 1.44)α = 46.4α (α from table 9.22)

e. Blast load, depends on time (table 9.3)

1 "b" edge of slab, $\left[1/2(2) = 1 \text{ "b" edge}\right]$

2 "a" edges of slab

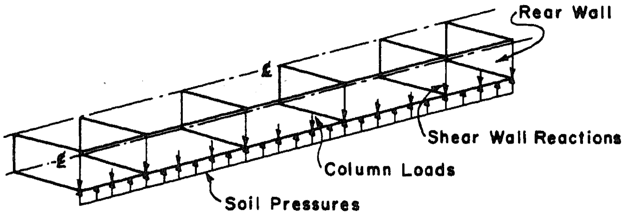
0.20 $\overline{P}_r$ (direct load on wall and girder = $\left[1.0(9.5) + 0.83(24.0)\right] \frac{\overline{P}_{roof}}{144}$ = 0.20 $\overline{P}_{roof}$)

f. Soil pressure from blast load multiplied by footing area (64.45 ft^2). (Pressures from table 9.24.)

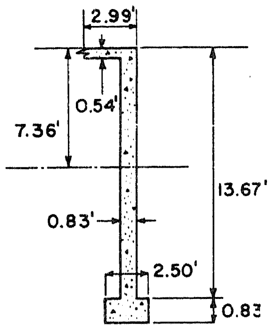
Forces (kips) / t (sec)	a	b	c	d	e: 1 "b" edge	e: 2 "a" edges	e: $0.20\overline{P}_{roof}$	e	f	Net Front Wall*	Net Back Wall**
0	64.0	58.60	0	4.4	0	0	0	0	0	10	0
0.0025	64.0	58.60	0	12.5	2.40	2.34	0.12	4.06	9.7	11	13
0.005	64.0	58.60	1.3	26.0	6.27	5.84	0.24	12.35	22.6	21	31
0.0075	64.0	58.60	3.9	46.8	12.68	11.38	0.36	24.42	41.2	39	63
0.010	64.0	58.60	9.7	68.2	21.58	19.84	0.50	41.92	65.0	60	96
0.0125	64.0	58.60	20.0	86.6	32.05	27.68	0.62	60.35	92.0	80	132
0.015	64.0	58.60	35.4	90.4	42.55	36.04	0.73	79.32	118.0	103	159
0.0175	64.0	58.60	56.8	78.0	51.80	43.36	0.86	96.02	142.5	94	177
0.020	64.0	58.60	83.2	49.2	58.40	49.50	1.00	108.9	159.0	87	171
0.0225	64.0	58.60	113.0	8.8	62.05	52.90	1.12	116.07	174.0	68	173
0.025	64.0	58.60	143.0	-32.8	64.80	55.40	1.23	121.4	184.0	52	168
0.0275	64.0	58.60	170.0	-66.7	87.60	57.80	1.35	146.8	195.0	59	147
0.030	64.0	58.60	193.0	-84.0	71.60	61.80	1.49	134.9	210.0	44	180
0.0325	64.0	58.60	209.0	-85.4	73.30	67.60	1.71	147.61	228.0	48	196
0.035	64.0	58.60	224.0	-76.5	86.70	74.60	1.73	163.03	250.0	66	225
0.0375	64.0	58.60	232.0	-66.2	91.30	78.40	1.72	171.42	260.0	82	250
0.040	64.0	58.60	235.0	-66.7	93.20	78.00	1.69	170.89	262.0	84	259†
0.0425	64.0	58.60	234.0	-80.7	94.80	81.20	1.67	177.67	266.0	80	237
0.045	64.0	58.60			95.70	82.04	1.66				

* (a + c + d + e - b - f)
** (b + c + d + f - a - e)
† Maximum

b. Determination of Maximum Vertical Force on Walls. The maximum net vertical force occurs at the back wall at $t = 0.040$ sec from table 9.26. In order to determine the maximum shear and moments in the wall a moment distribution will be performed using properties obtained below.



c. Properties of Wall Section.



Rear Wall. Computation of I and Area

$b' = 4t + b = 4(0.54) + 0.83 = 2.99$ ft

$$\bar{y} = \frac{(2.16)0.54(0.27) + (0.83)13.67(6.84) + (2.5)0.83(14.08)}{2.16(0.54) + 0.83(13.69) + 2.5(0.83)}$$

$$\bar{y}_{top} = \frac{107.11}{14.56} = 7.36 \text{ ft}$$

$$A_w = 0.83(14.50) = 12.00 \text{ ft}^2$$

$$I = (1/12)\ 2.16(0.54)^3 + (0.54)\ 2.16(7.09)^2 + (1/12)\ 0.83(13.67)^3 + (11.32)(0.52)^2 + (1/12)\ 2.5(0.83)^3 + (2.08)(6.72)^2 = 0.0283 + 58.4 + 1{,}762.0 + 4.22 + 0.143 + 93.6$$

$$I = 1{,}918.3 \text{ ft}^4$$

d. Constants for Moment Distribution--Center Spans (Par. 9-04).

$a_1 = 1,\ a_2 = 0.5,\ a_3 = 0.333$

$$S = \frac{I_o}{L^2 A_o}\frac{E}{G} = \frac{(1{,}918)\ 2.2}{(36.0)^2\ 12.0} = 0.272\ ;\ E/G = 2.2$$

$$a_3' = a_1 S + a_3 = 0.272 + 0.333 = 0.605$$

$$C_2 = \frac{a_2 - a_3'}{a_1 a_3' - (a_2)^2} = \frac{0.5 - 0.605}{0.605 - (0.5)^2} = -0.296$$

$$C_1 = \left(\frac{a_3'}{a_2 - a_3'}\right) C_2 = \left(\frac{0.605}{0.5 - 0.605}\right)(-0.296) = 1.703$$

$$K = \frac{C_1}{4}\frac{I_o}{L} = \frac{1.703}{4}\left(\frac{1{,}918}{36.0}\right) = 22.6$$

$$C.O. = \frac{C_2}{C_1} = \frac{-0.296}{1.703} = -0.174$$

$$\text{Modified K (symmetrical load)} = K\,(1 - C.O.)$$
$$= 22.6\,(1 + 0.174) = 26.5$$

e. <u>Constants for Moment Distribution--End Spans (Par. 9-04).</u>

$a_1 = 1,\ a_2 = 0.5,\ a_3 = 0.333$

$$S = \frac{(1{,}918)\ 2.2}{(24.0)^2\ 12.0} = 0.610\ ;\ E/G = 2.2$$

$$a_3' = 0.610 + 0.333 = 0.943$$

$$C_2 = \frac{0.5 - 0.943}{0.943 - (0.5)^2} = -0.640$$

$$C_1 = \left(\frac{0.943}{0.5 - 0.943}\right)(-0.640) = 1.360$$

$$K = \frac{1.360}{4}\left(\frac{1{,}918}{24.0}\right) = 27.2$$

$$C.O. = \frac{-0.640}{1.360} = -0.470$$

$$\text{Modified K (end span)} = K\left[1 - (C.O.)^2\right] = 27.2\left[1 - (-0.470)^2\right] = 21.2$$

f. <u>Load on 24-ft Span of Wall.</u> (Time = 0.040 sec)

Concentrated load:

Column load:	Slab	62.0 ft^3 concrete	
(gravity)	Girder	19.15	
	Column	12.17	
		93.32 ft^3 (0.150) =	14.0 kips
(blast)	1 "b" edge (from table 9.26)	=	93.20
	directly on girder $\frac{144(9.5)\ 1.0\ \bar{P}_{roof}}{1{,}000}$		
		= 0.067(8.45) =	0.56
		Total load on column	107.76 kips

Assume attached column footing pressure as a concentrated load to be subtracted from concentrated load on column.

Area footing = (2.67)(1.67) = 4.40 ft^2

Net soil pressure at t = 0.040 is 8.63 ksf (table 9.24)

Upward pressure under column = (4.40)(8.63) = 38.00 kips

Net concentrated load = 107.76 - 38.00 = 69.76 kips

Uniformly distributed load:

Soil pressure = (8.63)(2.50)(24.0)	= 520.0 ↑
Gravity load = 64.0 - 14.0	= 50.0 ↓
Blast load = 2 "a" edges + (1.69 - 0.56)	= 77.1 ↓
Inertial force	= 49.6 ↓

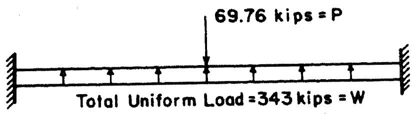
Net uniform load = 343.3 kips ↑

g. Analysis of Deep Beam Using Moment Distribution.

Fixed-end moments for 24-ft span:

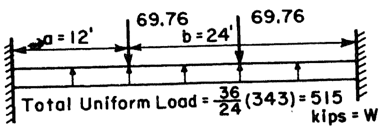
$$\text{Unif FEM} = \frac{1}{12} WL = \frac{(343)24}{12} = 686 \text{ ft-kips}$$

$$\text{Conc FEM} = \frac{PL}{8} = \frac{(69.76)24}{8} = -209 \text{ ft-kips}$$



Net FEM = +477 ft-kips

Fixed-end moments for 36-ft span:

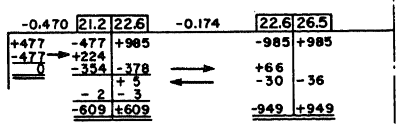


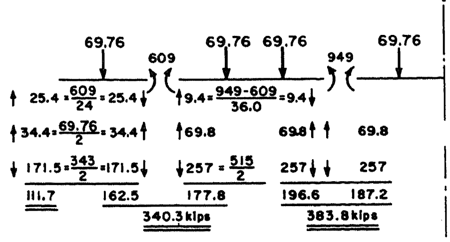
$$\text{Unif FEM} = \frac{1}{12} WL = \frac{(515)(36)}{12} = +1{,}542$$

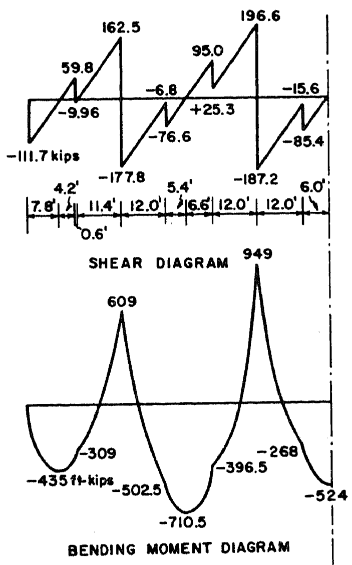
$$\text{Conc FEM} = \frac{P\left[ab^2 + a^2b\right]}{L^2} = \frac{Pab(b + a)}{L^2}$$

$$= \frac{Pab}{L} = \frac{69.76(12)\ 24}{36} = -557 \text{ ft-kips}$$

Net FEM = 1,542 - 557 = 985 ft-kips







h. Shear Strength.

Maximum V = 196.6 kips (back wall)

$$v = \frac{V}{bjd} = \frac{196.6(1{,}000)}{(0.83)13.5(144)} = 122 \text{ psi} < 390 = 0.10\, f'_{dc} \quad \therefore \text{ OK}$$

Maximum reaction = 383.8 kips (shear wall)

$$v = \frac{V}{bjd} = \frac{383.8(1{,}000)}{(0.83)13.5(144)} = 237 \text{ psi} < 390 = 0.10\, f'_{dc} \quad \therefore \text{ OK}$$

i. Steel Required for Bending.

Maximum moment = ± 949 ft-kips (depending upon direction of blast)

$$A_s = \frac{M}{f_{dy} d} = \frac{(949.0)}{52(13.50)} = 1.35 \text{ in.}^2$$

At top and bottom each 3 #6 bars, $A_s = 1.32$ in.^{2}

$\Sigma o = 7.1$ in.

$$u = \frac{V}{\Sigma o jd} = \frac{196.6(1{,}000)}{7.1(0.875)\ 13.25(12)} = 200 \text{ psi.} \quad 200 < 450 = 0.15\, f'_c \quad \text{OK}$$

The steel requirement for other sections of the wall can be determined in proportion to the moment, as indicated in the moment diagram on page 93.

9-18 FINAL DESIGN OF SHEAR WALLS. a. Investigation of Moment and Shear at Opening - Interior Shear Wall. Using the results obtained from the moment distribution analysis (par. 9-14) and the dynamic sliding and overturning analysis (par. 9-15) the moment and shear in the wall at the top and bottom of the opening for the maximum lateral shear wall deformation will be determined.

b. Top of Opening (Member AB, Par. 9-14p).

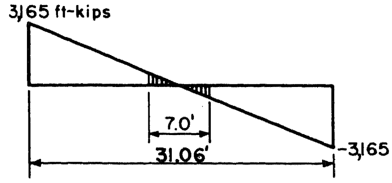
Moment = $1.17(10)^6$ ft-kips/ft of deflection (par. 9-14i)

Maximum deflection = $235.9(10)^{-5}$ ft (table 9.23)

Ratio of actual maximum load on wall (from roof deep-beam slab analysis) to assumed load on wall (uniformly distributed for rigid body assumption) = $\frac{931.2}{810.0} = 1.149$

Maximum moment at end of member AB = $1.17(10)^6\ 235.9(10)^{-5}\ (1.149)$
= 3,165 ft-kips

Moment at face of opening = $\frac{7.0}{31.06}$ (3,165)
= 715 ft-kips



MOMENT DIAGRAM—MEMBER AB

$$A_s = \frac{M}{f_{dy}d} = \frac{715}{52(3.5)} = 3.92 \text{ in.}^2$$

Use 4 #9 bars in two rows (A_s = 4.00 in.^{2}, $p = \frac{4.00}{(10)3.5(12)}$ = 0.00952) at both top and bottom of lintel to resist moment due to blast from either side of building.

Shear throughout member (considering end moments only) = $\frac{\text{2 moment}}{\text{span}}$
$= \frac{2(3,165)}{31.06}$ = 204 kips

Shear intensity $v = \frac{V}{bjd} = \frac{204,000}{10(7/8)45}$ = 519 psi

Using stirrups, allowable shear stress = $0.04\ f'_c + 5{,}000p + rf_y$ (eq 4.24)

$519 = 0.04(3{,}000) + 5{,}000(0.00952) + r(40{,}000)$

$r = \frac{519 - 168}{40{,}000} = 0.00878$

Web reinforcement = r(10)12 = 0.00878(10)12 = 1.055 in.^{2}/ft

Use U stirrup #5 at 7 in. (A_s = 1.06 in.^{2}) throughout lintel.

c. Bottom of Opening (Member CD, Par. 9-14h).

Moment = $1.28(10)^6$ ft-kips/ft of deflection (par. 9-14i)

Maximum deflection = $235.9(10)^{-5}$ ft (table 9.23)

Ratio of loading on wall = 1.149

Moment at end of span = $1.28(10)^6\ 235.9(10)^{-5}\ 1.149$ = 3,475 ft-kips

Moment in member at face of opening = $\frac{7.0}{31.06}$ (3,475) = 782 ft-kips

$$A_s = \frac{M}{f_{dy}d} = \frac{782}{52(2.50)} = 6.0 \text{ in.}^2$$

Use 6 #9 bars in one row (A_s = 6.00 in.^{2}, $p = \frac{6.00}{30(30)}$ = 0.0067) at both top and bottom of member to resist moment due to blast from either side of building.

Shear throughout member (considering end moments only) = $\frac{\text{2 moment}}{\text{span}}$
$= \frac{2(3,475)}{31.06}$ = 224.0 kips

Shear intensity $v = \frac{V}{bjd} = \frac{224.0(1,000)}{30(7/8)33} = 258$ psi

Allowable shear stress $= 0.04\, f'_c + 5,000\, p + rf_y$ (eq 4.24)

$258 = 0.04(3,000) + 5,000(0.0067) + r(40,000)$

$r = \frac{258 - 153.5}{40,000} = 0.00262$

Area of shear reinforcement per foot of beam = 0.00262(30)12 = 0.944 in.^{2}

Use U stirrups #5 at 8 in. (A_s = 0.94 in.^{2})

d. Investigation of Moment and Shear at Opening - End Shear Wall. Ratio of actual maximum load on wall (because of deep-beam roof slab action investigation) to assumed load on wall (using shear wall stiffnesses and deflection from dynamic overturning and sliding analysis) $= \frac{239.8}{333.0}$ = 0.720 which is less than 1.0, therefore use full ratio, 1.0.

e. Top of Opening (Member AB, Par. 9-14g).

Moment = $1.02(10)^6$ ft-kip/ft of deflection (par. 9-14r)

Maximum deflection = $235.9(10)^{-5}$ ft (table 9.23)

Moment at end of member = $1.02(10)^6\ 235.9(10)^{-5}$ = 2,400 ft-kips

Moment at face of opening $= \frac{6.0}{29.0}(2,400) = 496$ ft-kips

$A_s = \frac{M}{f_{dy}d} = \frac{496}{52(2.50)} = 3.82$ in.^{2}

Use 4 #9 bars in two rows, $A_s = 4.00$ in.^{2}, $p = \frac{4.0}{(10)2.5(12)} = 0.0133$ at both top and bottom of lintel to resist moment due to blast from either side of building.

Shear throughout member $= \frac{2\ \text{moment}}{\text{span}} = \frac{2(2,400)}{29.0} = 165.5$ kips

Shear intensity $v = \frac{V}{bjd} = \frac{165,500}{10(7/8)45} = 423$ psi

Using stirrups, shear intensity $= 0.04\, f'_c + 5,000\, p + rf_y$ (eq 4-24)

$423 = (0.04)(3,000) + 5,000(0.0133) + r(40,000)$

$r = \frac{423 - 186.5}{40,000} = 0.00592$

Web reinforcement = r(10)12 = 0.710 in.^{2}

Use U stirrups - #5 at 10 in., A_s = 0.74, throughout lintel.

f. Bottom of Opening (Member CD, Par. 9-14n).

Moment = $0.89(10)^6$ ft-kips/ft of deflection (par. 9-14r)

Moment at end of span = $0.89(10)^6(235.9)(10)^{-5}$ = 2,100 ft-kips

Moment at face of opening = $\frac{6.0}{29.0}$ (2,100) = 434.0 ft-kips

$$A_s = \frac{M}{f_y d} = \frac{434}{52(2.50)} = 3.34 \text{ in.}^2$$

Use 8 #6 bars in one row, $A_s = 3.52$ in.^{2}, $p = \frac{3.52}{(30)30} = 0.0039$, at both top and bottom of member to resist moment due to blast from either side of building.

$$\text{Shear throughout member} = \frac{2 \text{ moment}}{\text{span}} = \frac{2(2,100)}{29.0} = 145.0$$

$$\text{Shear intensity} \quad v = \frac{V}{bjd} = \frac{145.0}{30(7/8)33} = 167.0 \text{ psi}$$

Allowable shear stress = $0.04\, f'_c + 5,000\, p + rf_y$ (eq 4-24)

(without stirrups) = $0.04(3,000) + 5,000(0.0039)$

= 139.5 psi

$167 > 139.5$ $\therefore$ use stirrups

$rf_y = 167.0 - 139.5 = 27.5$

$r = \frac{27.5}{20,000} = 0.00138$

Area of shear reinforcement per foot of beam = 0.00138(30)12 = 0.497

Use U stirrups #4 at 9 in., $A_s = 0.54$.

g. Shear Panel Reinforcement. Each shear wall is divided by the corridor opening into two shear wall panels jointed together at the opening. The shear wall reinforcement is calculated using the width and vertical span of the individual shear wall panels rather than the entire shear wall. In the dynamic overturning and sliding analysis the maximum cracking resistance of the shear walls was not developed. Hence the shear panel steel will be selected to give an ultimate resistance equal to the stress developed in the dynamic analysis.

h. Interior Shear Walls (Par. 9-14b).

Vertical span of shear panels = 11.71 ft = H

Width of shear panels = 20.5 ft

Length center to center of vertical edge steel = 20.5 - 2
= 18.5 ft = L

Maximum total shear wall resistance (par. 9-16b and f) = 931.2 kips

Maximum shear per panel = 931.2/2 = 465.6 kips = R_{du}

Use minimum steel uniformly distributed in wall, #3 bars at 9 in. center to center, each side, each way, A_s = 0.30 in.^{2}/ft, p = 0.0025

Refer to equation (4.55)

$P = f_{dy}pt(H + L) = 52(0.0025)\ 10(11.71 + 18.5)\ 12 = 471$ kips

$P/R_{du} = 471/466 = 1.01$

From figure 4.36, P/C = 1.60, $C/R_{du} = 0.63$

Req'd C = 0.63 R_{du} = 0.63(465.6) = 293 kips

$$C = A_s f'_{dc}\left[15 + 1.9\left(\frac{L}{H}\right)^2\right] = 293 = A_s(3.9)\left[15 + 1.9\left(\frac{18.5}{11.71}\right)^2\right]$$

$$\text{Req'd } A_s(\text{vertical edge steel}) = \frac{293}{3.9(19.75)} = 3.80 \text{ in.}^2$$

Use 4 #9 bars as edge steel for shear panels, A_s = 4.00 in.^{2}

i. <u>End Shear Walls (Par. 9-14k).</u>

Vertical span of shear panels = 11.45 ft = H

Width of shear panels = 21.0 ft

Length center to center of vertical edge steel = 21.0 - 2
= 19.0 ft = L

Maximum total shear wall resistance (par. 9-16b and f) = 333 kips

Maximum shear per panel = 333/2 = 167 kips = R_{du}

Use minimum steel uniformly distributed in wall. Vertical steel is already present for bending under normal to plane of wall.

Use #3 bars at 9 in. center to center, each side, horizontally, A_s = 0.30 in.^{2}/ft, p = 0.0025

Refer to equation (4.55)

P = 52(0.0025) 10(11.45 + 19.0) = 396 kips

$P/R_{du} = 396/167 = 2.37$

From figure 4.36, P/C > 3, $C/R_{du} \approx 0.55$

$$C = A_s(3.9)\left[15 + 1.9\left(\frac{19.0}{11.45}\right)^2\right] = 0.55(167) = 91.8 \text{ kips}$$

Req'd A_s(vertical edge steel) = $\frac{91.8}{3.9(20.24)}$ = 1.17 in.^{2}

Use 2 #7 bars as edge steel for shear panels, A_s = 1.20 in.^{2}

9-19 DESIGN SUMMARY. Figure 9.22 shows the location of the various elements that were designed in this example. Sections of the various elements

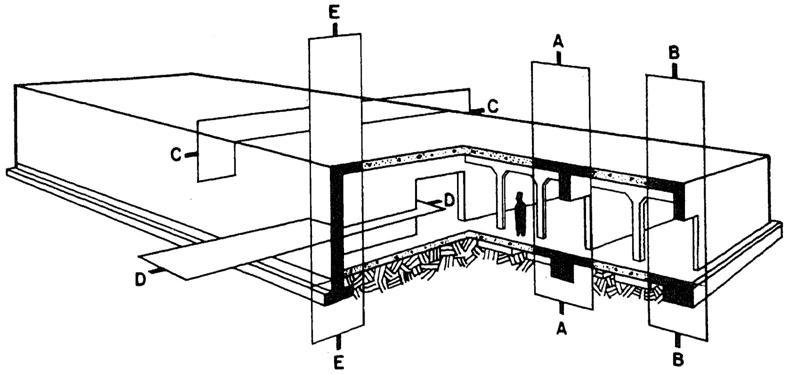


Figure 9.22. Locations of designed sections

are shown in figures 9.22a through 9.22e to indicate the final design. References are given on each section to the paragraphs where each design is first presented. The steel reinforcement is not shown in detail because the example is limited to the principle design considerations. Only the main structural reinforcement as computed in the design calculations is illustrated.

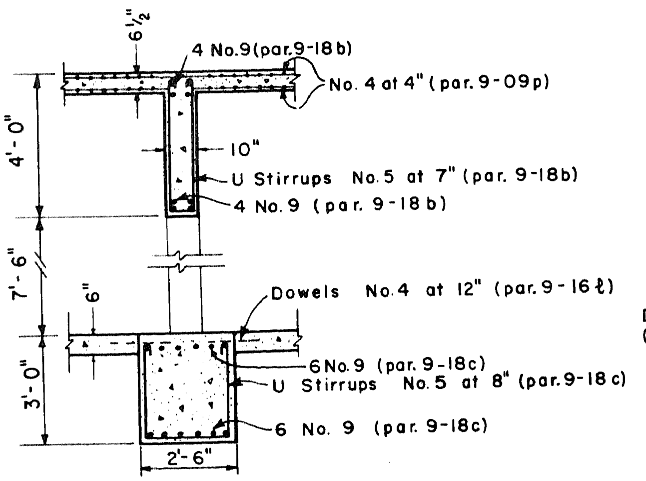


Figure 9.22a. Section A-A of figure 9.22

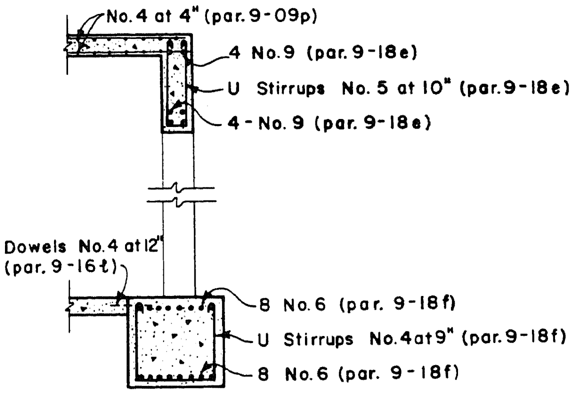


Figure 9.22b. Section B-B of figure 9.22

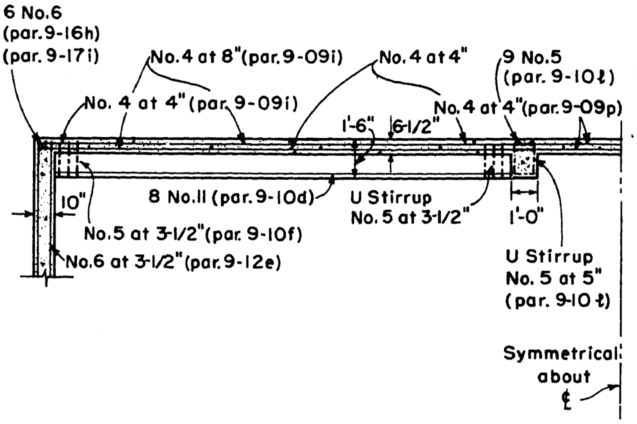


Figure 9.22c. Section C-C of figure 9.22

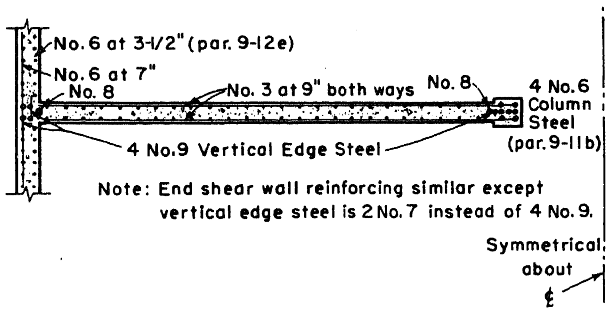


Figure 9.22d. Section D-D of figure 9.22

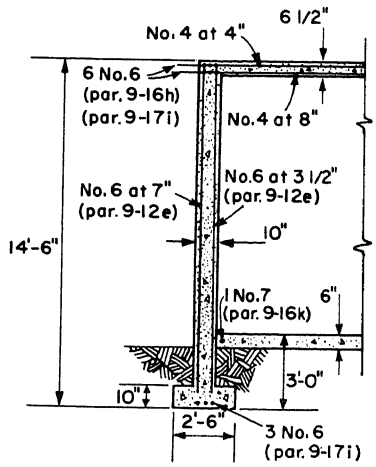


Figure 9.22e. Section E-E of figure 9.22

DESIGN EXAMPLE - TWO-STORY SHEAR WALL BUILDING

9-20 GENERAL. a. Statement of Problem. The design of a windowless two-story reinforced concrete shear wall building is presented to illustrate the principles discussed in paragraph 9-04 to 9-07. The plan of the building is shown in figures 9.23 and 9.24. Typical sections through the building are shown in figures 9.25 and 9.26. The roof construction is identical with that of the one-story shear

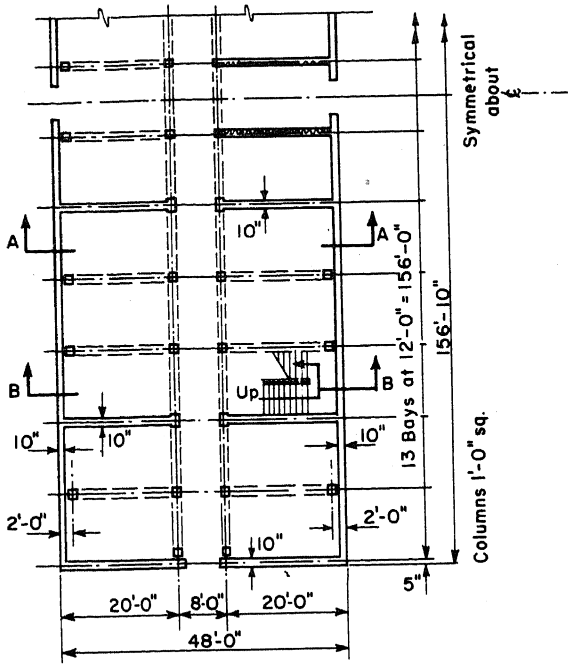


Figure 9.23. Plan of two-story reinforced concrete shear wall building

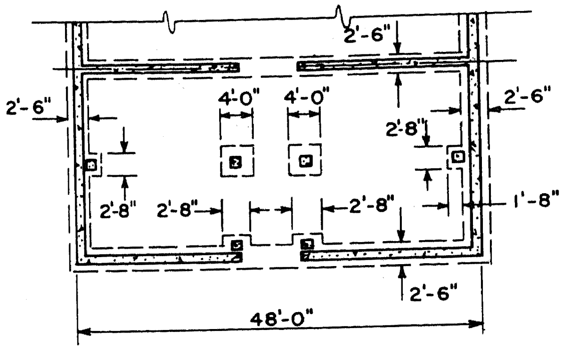


Figure 9.24. Typical foundation arrangement

wall example presented previously, having a two-way slab over the office areas and a one-way slab over the corridor. The location of the shear walls was chosen to suit the functional requirements of the building. The second floor slab is supported by girders similar to the roof. The design of the second floor members is not presented since they need be designed for the conventional floor loads only.

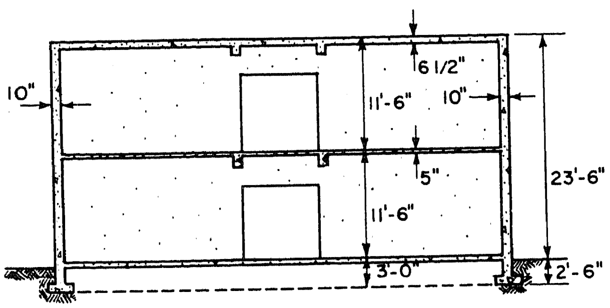


Figure 9.25. Section A-A of figure 9.23 at shear wall

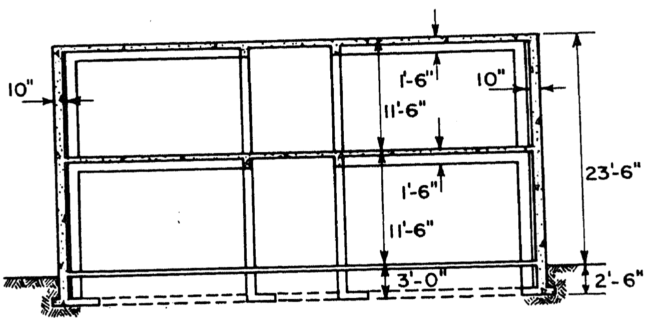


Figure 9.26. Typical section B-B of figure 9.23

The air blast incident overpressure vs time curve which the building is designed to resist is plotted in figure 9.27. The air blast may approach the structure from any direction. The shock front is normal to the ground surface. The peak air blast overpressure is 10 psi and the duration of the positive phase is 0.71 sec (fig. 3.10).

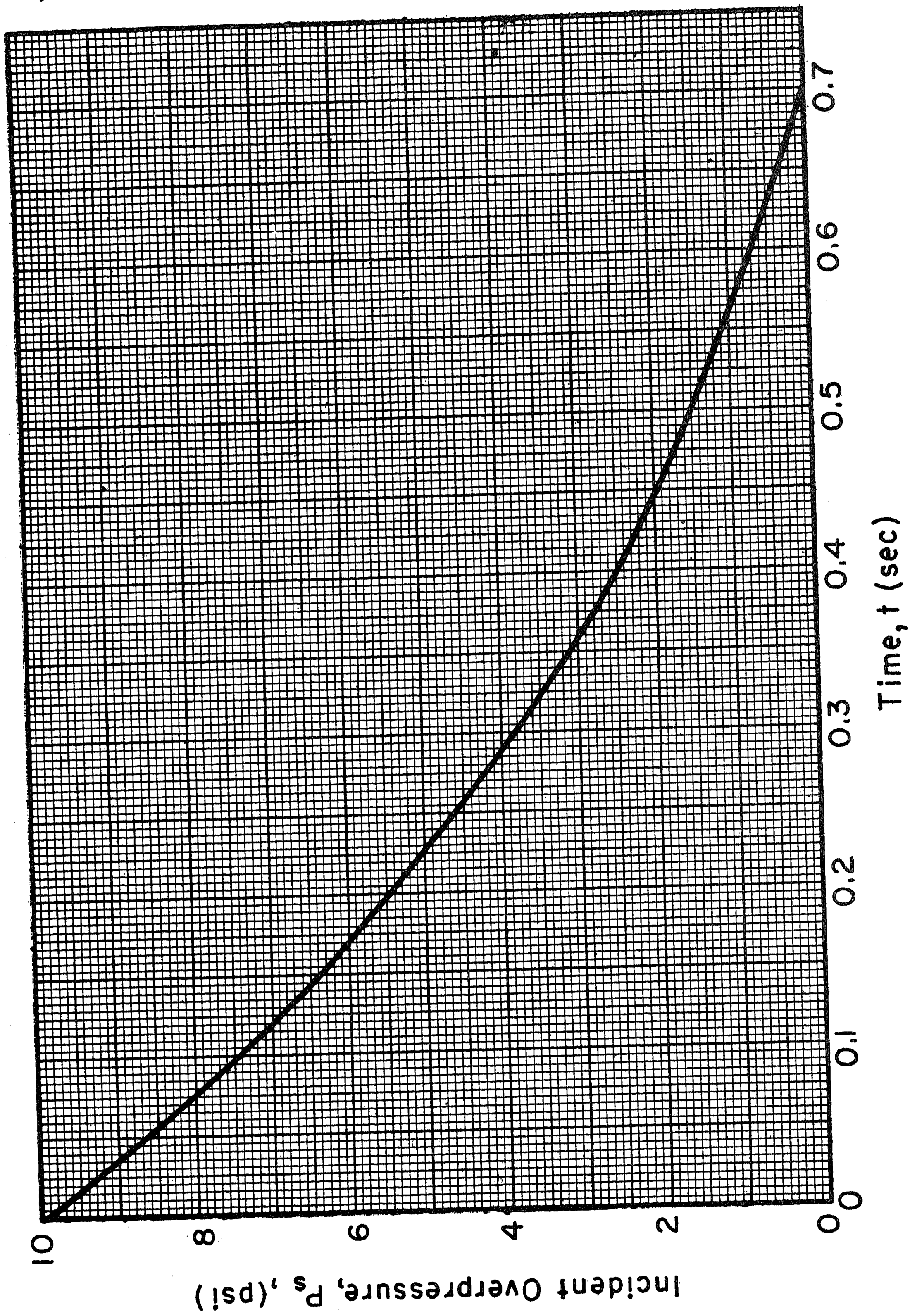


Figure 9.27. Air blast incident overpressure vs time curve

The strength properties of the materials to be used are given below. Intermediate grade reinforcing steel and a 3,000-psi concrete are to be specified. A 30 percent increase in strength is used because of the dynamic character of the applied loads. The notation used is that introduced in EM 1110-345-414. It will be noted that the strength values assumed for this example do not agree with the values recommended in EM 1110-345-414. The latter should be used for current projects.

$f_y = 40,000$ psi
$E_c = 3(10)^6$ psi
$f_{dy} = 52,000$ psi
$f_{dc} = 3,900$ psi
$f'_c = 3,000$ psi
$n = 10$

The structure is to be located upon a compact sand-gravel mixture having the following properties (par. 4-15).

Normal load-bearing capacity = 10 kips/sq ft
Ultimate load-bearing capacity = 30 kips/sq ft
Modulus of elasticity = 40,000 psi
Coefficient of friction (soil on soil) = 0.75
Coefficient of friction (concrete on soil) = 0.75
Unit weight of soil = 100 lb/ft^3
Normal component of passive pressure coefficients $K_{P\phi} = 10$

b. Design Procedure. The design of the structural elements is accomplished in the following order in accordance with the procedures presented in paragraph 9-07.

9-21 Design of Roof Slabs and Roof Girders
9-22 Column and Column Footing Design
9-23 Wall Slab Design (Maximum Normal Loading)
9-24 Rigid Body Overturning and Sliding Analysis
9-25 Determination of Shear Wall Resistance Functions
9-26 Shear Wall Analysis
9-27 Simultaneous Dynamic Overturning and Sliding Investigation
9-28 Roof and Floor Slab Design (Deep Beam Action)
9-29 Wall Analysis (Deep Beam Action)
9-30 Final Design of Shear Walls
9-31 Design Summary

9-21 DESIGN OF ROOF SLABS AND ROOF GIRDERS. a. Vertical Blast Loads on

<u>Roof.</u> The direction of travel of the air blast wave is assumed to be normal to either wall of the building to determine the critical load for all elements. Paragraph 3-09 indicates the variations in roof loading with direction of the blast wave by the use of zones where:

Zone 1 indicates full incident blast wave overpressure.

Zone 2 indicates slightly reduced incident blast wave overpressure.

Zone 3 indicates greatly reduced incident blast wave overpressure.

With the direction of travel of the blast wave normal to the long dimension of the building, it is apparent that the roof loading in the end bays (Zones 1 and 2) is greater than in the center bays (Zone 3), and the end roof panels are critical. However, with the air blast wave traveling in the direction normal to the short dimension of the building, the maximum loading (Zone 1) exists over the entire roof. Therefore, the roof slabs must be all designed to carry the Zone 1 loading which is the full incident blast wave overpressure illustrated in figure 9.27.

For the portion of the roof designed as a series of two-way slabs, it is necessary to consider the total load on each slab. The total load for a typical slab is obtained by using the average Zone 1 pressure on the slab computed as explained in paragraph 3-09e. This average overpressure-time curve is plotted in figure 9.28.

For the portion of the roof designed as a one-way slab, it is necessary to consider only the instantaneous pressure on a one-foot strip of slab using the local Zone 1 overpressure-time curve (fig. 9.27).

It is important to note that the average blast loading over the entire roof surface which takes into account the reduction of overpressure in Zone 3 (fig. 9.29) is used in the foundation design and overturning and sliding analysis because it gives the average total load on the roof. To assume that the average Zone 1 loads act over the entire roof would not be realistic for the sliding analysis because the total load and therefore the friction would be too great.

b. <u>Design of Two-way Roof Slab.</u> The roof slab design in this step is based on only the blast loads normal to the slab. The deep beam action of the roof slab acting as a horizontal load-carrying element is temporarily disregarded because the lateral loads cannot be obtained until after

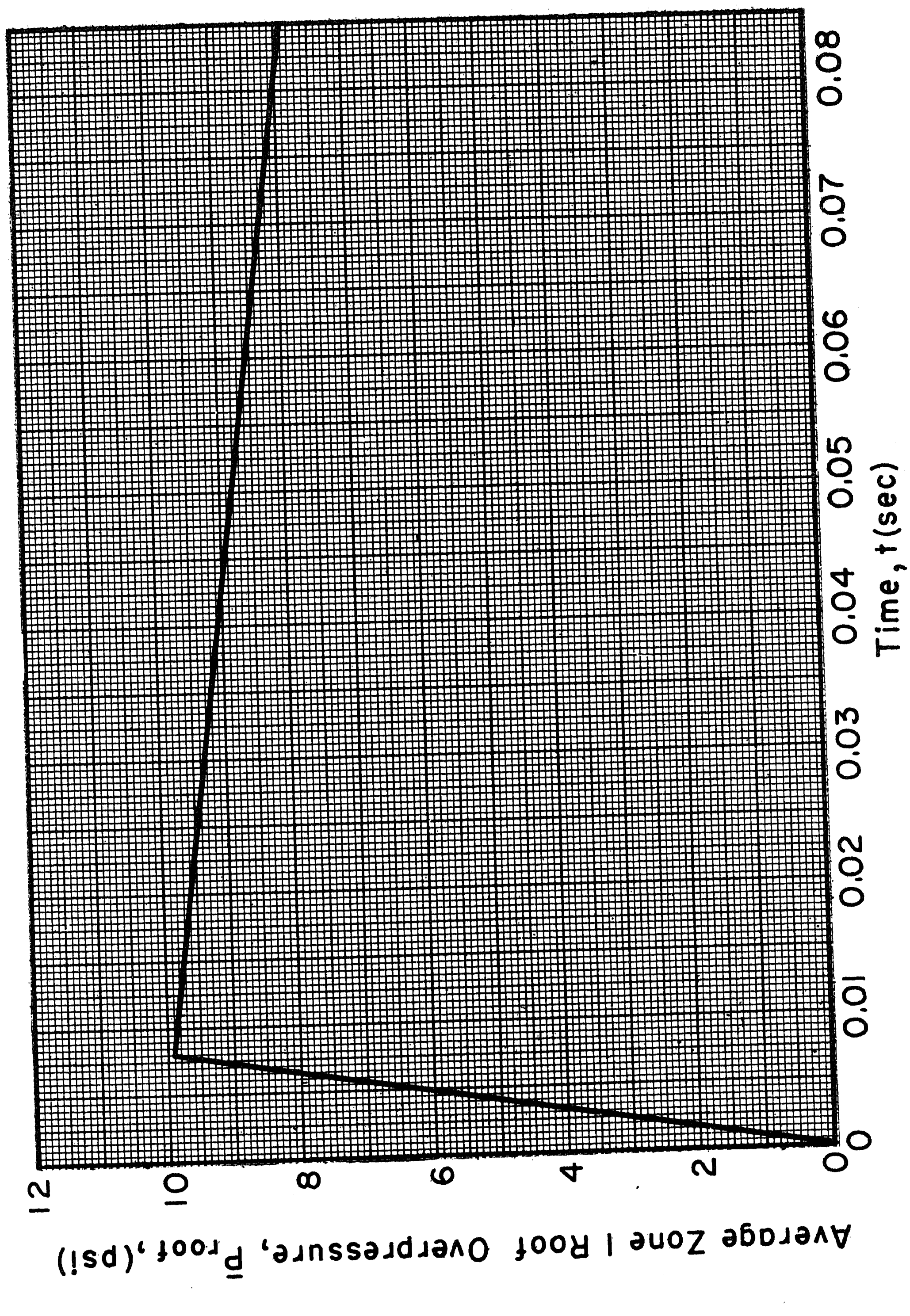


Figure 9.28. Average Zone 1 roof overpressure-time curve on 11-ft-1-in. by 18-ft-8-in. slab

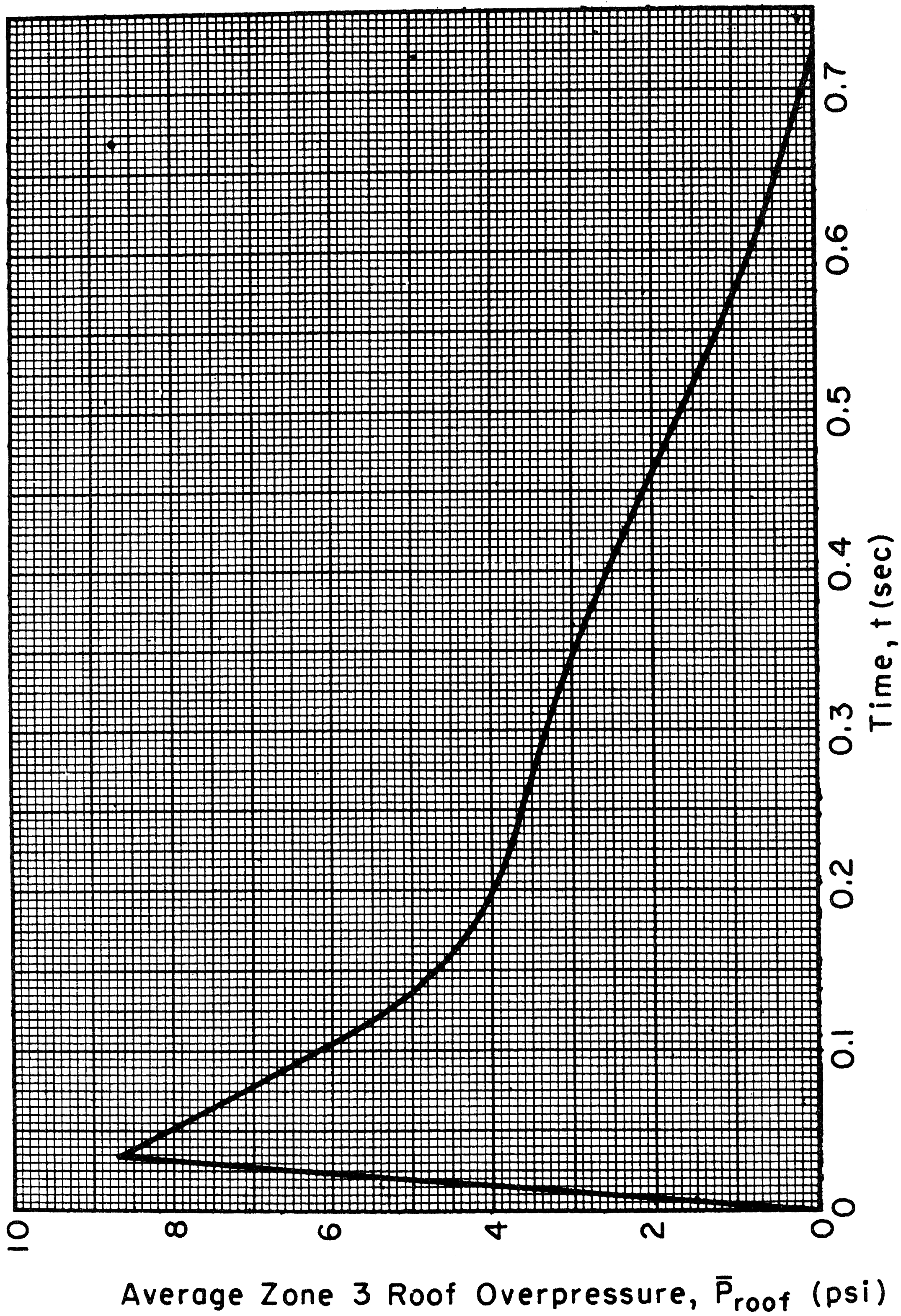


Figure 9.29. Average Zone 3 roof overpressure-time curve

the front wall has been designed. (This will be considered later in paragraph 9-23.) The design is based on elastic and elasto-plastic action of a two-way slab so that additional capacity to absorb energy will be available in the purely plastic range. This will provide a reserve capacity against possible overloading of the slab due to the simultaneous deep beam action.

c. Design Conditions. As stated in paragraph 9-21a, the vertical blast load is assumed to be uniformly distributed over the slab. The continuity of the slab over the girders and walls justifies the assumption of full fixity at the four edges of the slab.

d. Slab Properties. Since the average Zone 1 roof overpressure is the same as in the preceding one-story design example, it will be possible to utilize the design in paragraph 9-09i, Revised Slab Properties. This consists of a 6-1/2-in. slab with #4 at 4 in. $\Sigma o = 4.7$ in., $A_s = 0.60$ in.^{2}/ft

Negative d = 4 in., negative p = (0.60)/4(12) = 0.0125

Positive d = 4.5 in., positive p = (0.60)/4.5(12) = 0.0111

Use #4 at 8 in. for positive steel at support and for negative steel at center of span for reverse bending resistance.

The dynamic reactions to be used in the foundation design and overturning analyses will be computed in table 9.27, using average Zone 3 roof overpressure-time curve (fig. 9.29).

e. Reaction Determination by Numerical Integration.

Use acceleration impulse extrapolation method (par. 5-08d)

$$y_{n+1} = 2\,y_n - y_{n-1} + \ddot{y}_n\,(\Delta t)^2 \quad \text{(eq 5.49)}$$

$$\ddot{y}_n = (P_n - R_n)/K_{LM}m_t$$

$$\Delta t = 0.0025 \text{ sec (approximately 1/10 elastic } T_n)$$

Use properties for slab (par. 9-09i)

Elastic strain range:

$$(\Delta t)^2/K_{LM}m_t = (0.0025)^2/0.71(0.521) = 0.0000169$$

$$\ddot{y}_n(\Delta t)^2 = 0.0000169\,(P_n - R_n)$$

$$R_n = k_1 y_n = 22{,}400\,y_n\,, \quad (y_n \le 0.0115 = y_e)$$

$$P_n = 29.8\,\bar{P}_s \text{ kips}$$

$\bar{P}_s$ is obtained from figure 9.29

$$V_A = 0.06P + 0.09R,\quad V_B = 0.12P + 0.23R$$

Table 9.27. Determination of Maximum Deflection and Dynamic Reactions for Two-way Roof Slab (Zone 3 Loading)

t (sec)	$\bar{P}_{roof}$ (psi)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y_n (ft)	Strain Range	V_A (kips)	V_B (kips)
0	0	0	0	18.46/6	0.000052	0	e	0	0
0.0025	0.62	18.0	1.16	17.30	0.000292	0.000052	e	1.12	2.48
0.005	1.25	37.3	8.86	28.44	0.000481	0.000396	e	3.03	6.51
0.0075	1.90	56.7	27.35	29.35	0.000495	0.001221	e	5.86	13.10
0.010	2.55	76.0	56.90	19.10	0.000322	0.002541	e	9.69	22.22
0.0125	3.18	94.8	93.60	1.20	0.000020	0.004182	e	14.12	32.80
0.015	3.80	113.0	131.0	-18.0	-0.000304	0.005843	e	18.58	43.20
0.0175	4.45	132.5	161.0	-28.5	-0.000482	0.007200	e	22.46	53.0
0.020	5.10	152.0	181.0	-29.0	-0.000490	0.008075	e	25.43	59.9
0.0225	5.72	170.5	189.5	-19.0	-0.000321	0.008460	e	27.35	64.2
0.025	6.35	189.0	191.0	-2.0	-0.000034	0.008524	e	28.55	66.7
0.0275	7.00	209.0	192.0	17.0	+0.000288	0.008554	e	29.82	69.3
0.030	7.62	227.0	199.0	28.0	+0.000474	0.008872	e	31.50	73.0
0.0325	8.30	247.0	217.0	30.0	+0.000507	0.009664	e	34.30	79.6
0.035	8.60	256.0	246.0	+10.0	+0.000169	0.010963	e	37.55	87.5
0.0375	8.50	253.0	263.7	-10.7	-0.000174	0.012431	e-p	39.1	91.2
0.040	8.45	252.0	271.9	-19.9	-0.000336	0.013725	e-p	40.0	93.2
0.0425	8.30	247.0	277.5	-30.5	-0.000495	0.014683	e-p	40.3	94.4
0.045	8.20	244.0	280.4	-36.4	-0.000590	0.015146	e-p	40.5	94.8
0.0475	8.10	241.0	277.4*	-36.4	-0.000615	0.015019	e	39.3	92.8
0.050	8.05	240.0	261.4*	-21.4	-0.000362	0.014277	e	37.9	88.2
0.0525	8.00	238.5	235.4*			0.013173	e	35.8	83.5
0.055	7.92	236.0						35.4	82.6
0.0575	7.82	233.0						35.0	81.5
0.060	7.75	231.0						34.7	81.0
0.0625	7.65	228.0						34.2	79.8
0.065	7.55	225.0						33.8	78.6

$$* R_n = 188 + 6{,}100\,(0.015146) - 22{,}400\,(0.015146 - y_n)$$
$$= 280.4 + 22{,}400\, y_n - 340$$
$$= 22{,}400\, y_n - 59.6$$

Elasto-plastic strain range:

$$(\Delta t)^2/K_{LM}m_t = (0.0025)^2/0.74(0.521) = 0.0000162$$

$$\ddot{y}_n(\Delta t)^2 = 0.0000162(P_n - R_n)$$

$$R_n = 258 + 6,100\ (y_n - y_e)$$

$$= 188 + 6,100\ y, \quad (y_e = 0.0115 \leq y_n \leq 0.0620 = y_p)$$

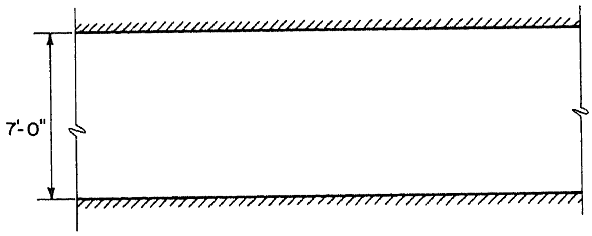
P_n is same as elastic case

$$V_A = 0.04P + 0.11R, \quad V_B = 0.09P + 0.26R$$

f. Design of One-way Roof Slab. The design of the one-way roof slab over the corridor is based on elastic and elasto-plastic action of a one-way slab so that additional capacity to absorb energy will be available in the purely plastic range to act as a safety factor against possible weakening of the slab due to simultaneous deep beam action. (Investigated later in par. 9-27.)

The load is assumed to be uniformly distributed over a one-foot strip of slab. The continuity of the slab over the girders and adjacent two-way slabs justifies the assumption of full fixity at the two edges of the slab. By the nature of the one-way action of the slab and the orientation in the building, the critical blast load will be the instantaneous loading of a one-foot strip with the local Zone 1 roof overpressure vs time curve when the air blast is traveling in a direction normal to the end walls of the structure.

g. Design Conditions. One-way slab, fixed at supports, elastic and elasto-plastic action, uniformly distributed load, equal strength at center and supports, Zone 1 loading condition.



Because Zone 1 loading for this structure is identical with the Zone 1 loading curve of the one-story shear wall structure presented in paragraph 9-08, the identical slab will be used. The computations for this design are presented in paragraphs 9-09p and 9-09q.

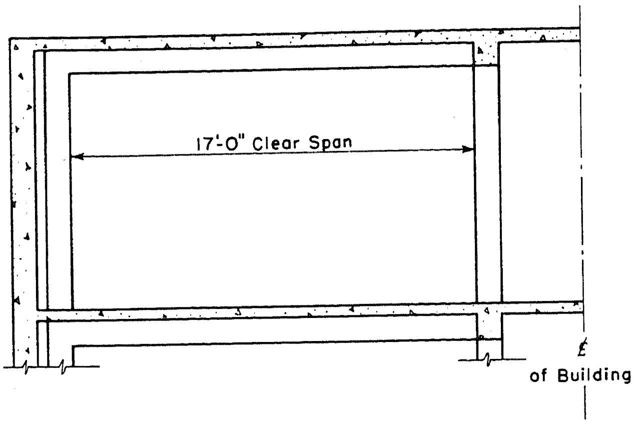
h. Slab Properties. Use t = 6-1/2 in. #4 at 4 in.; Σo = 4.7 in.; A_s = 0.60 in.^{2}/ft, d = 4.5 in., p = 0.0111.

Use same positive and negative steel throughout span to provide

reverse bending strength for shear wall action. Tension steel only is computed in strength and stiffness of slab.

Under the average Zone 3 loading, for overturning and sliding analyses, the slab behavior is assumed to be static because of its extremely short period, hence does not require a dynamic analysis in order to determine reactions.

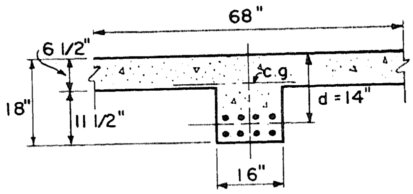
i. Design Condition for Transverse Roof Girders. Simple span T-beam, elastic and plastic action, uniformly distributed mass and roof slab reaction under Zone 1 loading condition; clear span = 17 ft 0 in.; $\frac{x_m}{x_e} = \alpha\beta = 6$ (par. 6-26).



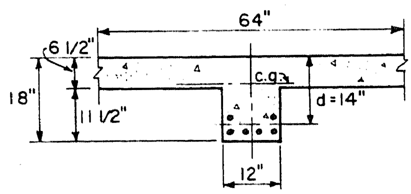
j. Girder Properties. Since the design loading for the girder is obtained from the Zone 1 reactions of two-way roof slabs, this girder is identical with that designed in detail in paragraph 9-10d.

Use 8 #11 bars, $A_s = 12.48$ in.^{2}, $\Sigma o = 40.0$ in.

Use #5 U stirrups at 3-1/2 in. at ends of span of beam.



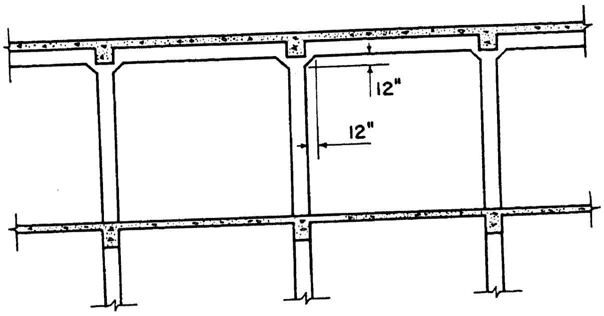
k. Design Condition for Longitudinal Roof Girders. Continuous span T-beam, elastic and elasto-plastic action, uniformly distributed mass and roof slab reactions under Zone 1 loading condition, clear span = 11 ft 0 in. Zone 1 loading for this structure is identical with one-story example, hence girder design will be similar (par. 9-10j).



l. Girder Properties. Positive steel at center of span: 6 #6 bars; $A_s = 2.64$ in.^{2}, p = 0.0157. Negative steel at support: 9 #5 bars; $A_s = 2.79$ in.^{2}, $\Sigma o = 17.6$ in. Use U stirrups #5 at 5 in. on each end of beam.

9-21m

m. Haunch on Longitudinal Girders. The design presented for longitudinal girders results in a relatively large shear stress at the intersection of longitudinal and transverse girders. The width of the transverse girder (16 in.) framing into the longitudinal girder will greatly increase this shear stress at the top of the column unless the column dimension in this direction is a minimum of 16 in. In order not to place this lower limit on column size a haunch will be used at each end of the longitudinal girders to receive the 16-in.-wide transverse girders framing into them. Haunches 12 in. along the column and 12 in. along longitudinal girders will be used. The actual depth of the section will be considered as resisting shear in accordance with ACI Code 702d in reference [9].



9-22 COLUMN AND COLUMN FOOTING DESIGN. a. Column Design Conditions. Axially loaded tied columns will be designed in accordance with the method in paragraph 4-11.

b. Design Loading. Using girder reactions from tables 9.5 and 9.7, a maximum column load G_4 is determined by summing the loads shown below in table 9.28.

Table 9.28. Column Loads (Zone 1 Loading)

G_1 = longitudinal girder reaction at time t_n (table 9.7)
G_2 = transverse girder reaction at time t_n (table 9.5)
G_3 = longitudinal girder reaction at time $t_n - t_n - 0.0075$ (table 9.7)
$G_4 = G_1 + G_2 + G_3$

t (sec)	G_1 (kips)	G_2 (kips)	G_3 (kips)	G_4 (kips)
0	0	0		0
0.0025	4.2	1.39		5.6
0.005	17.7	3.94		21.6
0.0075	44.9	9.76	0	54.7
0.010	74.7	16.78	4.2	95.7
0.0125	77.8	26.00	17.7	121.5
0.015	61.2	37.20	44.9	143.3
0.0175	38.9	52.20	74.7	165.8
0.020	40.9	69.40	77.8	188.1
0.0225	70.4	88.50	61.2	220.1
0.025	80.8	110.0	38.9	229.7
0.0275	74.5	131.0	40.9	246.4
0.030	47.4	146.2	70.4	264.0
0.0325	37.4	143.1	80.8	261.3
0.035	51.1	139.7	74.5	265.3*
0.0375	61.7	137.4	47.4	246.5

* Maximum dynamic column reaction.

Static column load: Consists of dead load of roof plus live and dead loads of second floor.

Roof:

Portion of two-way slab $= 11\left(\frac{11}{4}\right)\left(\frac{6.5}{12}\right) 150/1{,}000 = 2.5$ kips

Portion of one-way slab $= 11\left(\frac{7}{2}\right)\left(\frac{6.5}{12}\right) 150/1{,}000 = 3.1$ kips

Stem of girder $= 11(1.5)\ 150/1{,}000 = 2.5$ kips

Column load from longitudinal beam $= 8.1$ kips

Portion of two-way slab $= \frac{17}{2}(10.83)(0.54)\ \frac{150}{1{,}000} = 7.46$ kips

Stem of girder $= \left(\frac{17}{2}\right)1.33(1.5)\ \frac{150}{1{,}000} = 2.54$ kips

10.00 kips

Column load from roof $= 18.1$ kips

Second floor:

Occupancy load $= 12.0\left(\frac{20}{2} + \frac{8}{2}\right) 15.0^{*}/1{,}000 = 2.5$ kips

Dead load:

Portion of two-way slab $= 11\left(\frac{11}{4}\right)\left(\frac{5.0}{12}\right)\frac{150}{1{,}000} = 1.9$ kips

Portion of one-way slab $= 11\left(\frac{7}{2}\right)\left(\frac{5.0}{12}\right)\frac{150}{1{,}000} = 2.4$ kips

Stem of girder $= 11(1.5)\ \frac{150}{1{,}000} = 2.5$ kips

Portion of two-way slab $= \frac{17}{2}(10.83)(0.42)\ \frac{150}{1{,}000} = 5.8$ kips

Stem of girder $= \left(\frac{17}{2}\right)1.0(1.5)\ \frac{150}{1{,}000} = 1.9$ kips

Column load from second floor $= 17.0$ kips

Static column load (neglecting column weight) $= 18.1 + 17.0$

$= 25.1$ kips

---

* 15.0 lb/ft^2 is taken as the usual average occupancy loading on the first and second floors of the building, even though structural members were proportioned for 80 lb/ft^2. This load is applicable only for office and other administrative-type buildings as determined from investigations in reference [10], and applies to furniture, equipment, and people within the room.

Total maximum column reaction = 265.3 + 25.1 = 290.4 kips

Try column 12 in. square

$$P_p = 0.8\left[0.9(0.85)f'_{dc}\ A_c + A_s\ f_{dy}\right] \quad \text{(eq 4.27)}$$

Load taken by concrete = 0.8(0.9) 0.85(3.9) 144 = 341 kips

Use minimum steel, 4 #6 bars, A_s = 1.76 in.^{2}

Column capacity = 341 + 0.8(1.76) 52 = 412.2 kips > 290.4

Use column ties in accordance with ACI procedure.

c. Soil Pressure Investigation. Try 3-ft-6-in. by 3-ft-6-in. by 1-ft-2-in. column footing.

Total force on foundation:

Column load (par. 9-22b)	= 290.4 kips
Column weight = 0.15(1) 1(21.5)	= 3.2 kips
6-in. floor slab over footing = 0.15(3.5) 3.5(0.5)	= 0.9 kips
12-in. fill over footing = (0.12)(3.5) 3.5(1.0)	= 1.4 kips
Footing = 0.15(3.5) 3.5(1.2)	= 2.2 kips
	298.1 kips

Maximum static and dynamic bearing pressure = $\frac{298.1}{3.5(3.5)}$

= 24.4 kips/ft^2 < 30 kips/ft^2

The maximum bearing pressure is less than the ultimate load-bearing capacity of the foundation material, therefore the 3-ft-6-in. by 3-ft-6-in. footing is satisfactory.

d. Column Footing Design. Footing moment at face of column = 24.4(1.25)^2/2 = 19.1 kip-ft/ft. Footing shear at face of column = 24.4(1.25) = 30.5 kips/ft. Determine minimum depth of footing with 1.5 percent steel. From paragraph 9-09e using equation (4.17):

$M_p = 0.688d^2$ kip-ft/ft

$18.5 = 0.688d^2$, d = 5.2 in.

Try 14-in.-deep footing, d = 10 in. Try minimum p = 0.006

$$M_p = 0.006(52)\frac{12}{12}(10)^2\left[1 - \frac{(0.006)52}{1.70(3.9)}\right] = 29.8 \text{ kip-ft/ft}$$

29.8 > 19.1 OK

Allowable bond stress = $0.15f'_c$ (par. 4-09b) = 450 psi

Required Σo = $30.5(1{,}000)/450(12)\frac{7}{8}(10)$ = 0.65 in./ft

Required A_s = pbd = 0.006(12)10 = 0.72 in.^{2}/ft

Use #6 bars at 6 in. each way

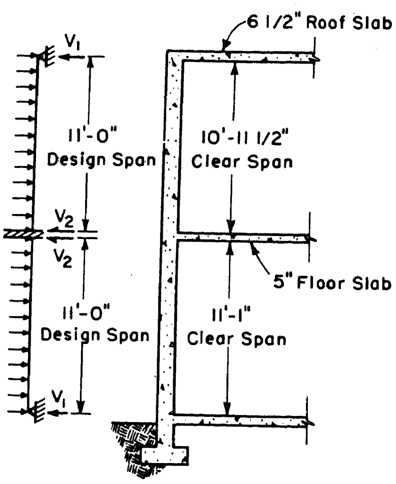
Furnished A_s = 0.88 in.^{2}/ft, Σo = 4.7 in./ft

e. Shear Strength. Total shear at distance "d" from faces of column $= 24.4\left[(3.5)^2 - (1 + 1.67)^2\right] = 125$ kips

Shear stress $= 125(1{,}000)/2.67(4)\frac{7}{8}(10)12 = 112$ psi

Allowable shear intensity (eq 4.24), $v_y = 0.04\,f'_c + 5{,}000\,p + rf_y$
$= 0.04(3.0) + 5{,}000(0.006) = 150$ psi > 112 psi $\therefore$ OK

9-23 WALL SLAB DESIGN (MAXIMUM NORMAL LOADING). a. Design Conditions.

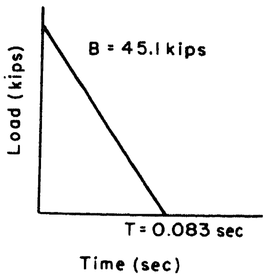


Spans fixed at second floor and pinned at roof and first floor; elastic, elasto-plastic, and plastic action; uniformly distributed load and mass; front wall loading condition; design for clear span of 11 ft 0 in. and use for first- and second-story walls; $x_m/x_e = \alpha\beta = 6$; minimum thickness = 10 in.

b. Design Loading. The design load as idealized from the computed loading shown by figure 9.30 is defined by

B = 45.1 kips

T = 0.083 sec



$$H = \frac{BT}{2} = \frac{(45.1)0.083}{2} = 1.87 \text{ kip-sec (par.6-11)}$$

c. Dynamic Design Factors. (Refer to table 6.1C.)

Elastic range:

$K_L = 0.58$ $\quad K_M = 0.45$ $\quad K_{LM} = 0.78$

$R_{1m} = \frac{8M_{Ps}}{L}$ $\quad k_1 = \frac{185EI}{L^3}$

$V_1 = 0.26R + 0.12P$ $\quad V_2 = 0.43R + 0.19P$

Elasto-plastic range:

$K_L = 0.64$ $\quad K_M = 0.50$ $\quad K_{LM} = 0.78$

$R_m = \frac{4}{L}(M_{Ps} + 2M_{Pm})$ $\quad k_{ep} = \frac{384EI}{L^3}$

$V = 0.39R + 0.11P$

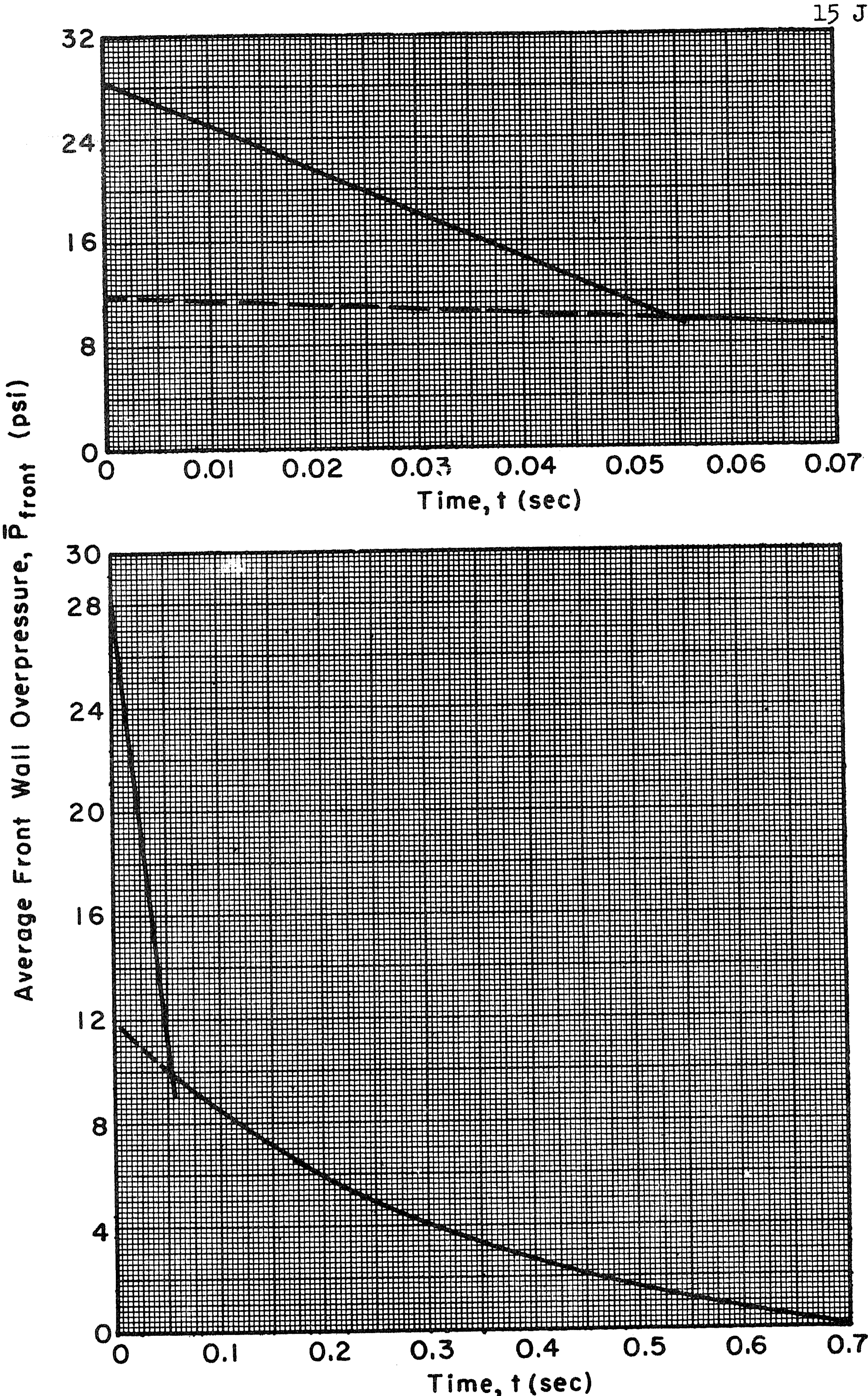


Figure 9.30. Average front wall overpressure vs time curve, $\overline{P}_{front}$ *(see par. 3-08a)*

Plastic range:

$K_L = 0.50$ $K_M = 0.33$ $K_{LM} = 0.66$

$$R_m = \frac{4}{L}(M_{Ps} + 2M_{Pm})$$

$$V = 0.38R + 0.12P$$

Average values:

$$K_L = \frac{0.64 + 0.50}{2} = 0.57$$

$$K_M = \frac{0.50 + 0.33}{2} = 0.42$$

$$R_m = \frac{4}{L}(M_{Ps} + 2M_{Pm})$$

$$k_E = \frac{160EI}{L^3}$$

d. First Trial - Actual Properties.

Let $M_{Ps} = M_{Pm} = M_P$

Assume $p = 0.015$ (par. 4-10)

Let $\alpha\beta = 6$ (par. 6-26)

Assume $C_R = 0.90$ (experience)

$$R_m = C_R B = 0.90(45.1) = 40.6 \text{ kips}$$

$$M_P = pf_{dy}bd^2\left(1 - \frac{pf_{dy}}{1.7f'_{dc}}\right) \quad \text{(eq 4.16)}$$

$$M_P = 0.015(52)(1)\,d^2\left[1 - \frac{0.015(52)}{1.7(3.9)}\right] = 0.688\,d^2 \text{ kip-ft (d in inches)}$$

$$R_m = \frac{12M_P}{L} = \frac{12(0.688)d^2}{11} = 40.6 \quad \therefore d = 7.35 \text{ in.}$$

Since the minimum wall thickness is 10 in. the percentage of steel can be reduced to give the desired resistance.

Letting $M_{Ps} = M_{Pm}$ and considering the center section

$$R_m = \frac{12M_{Pm}}{L} = 40.6 \qquad M_{Pm} = 37.2 \text{ kip-ft}$$

Assuming #5 bars

$$d = 10 - 0.75 - 0.31 = 8.94 \text{ in.}$$

$$M_{Pm} = p(52)(1)(8.94)^2\left[1 - \frac{p(52)}{1.7(3.9)}\right] = 37.2 \quad \therefore p \approx 0.01$$

Try $p = 0.0097$ at center

$$M_{Pm} = 0.0097(52)(1)(8.94)^2\left[1 - \frac{0.0097(52)}{1.7(3.9)}\right] = 37.2 \text{ kip-ft}$$

$$R_m = \frac{12M_{Pm}}{L} = \frac{12(37.2)}{11} = 40.6 \text{ kips}$$

$$I_g = \frac{1}{12} bh^3 = \frac{1}{12}(12)(10^3) = 1{,}000 \text{ in.}^4$$

$$I_t = bd^3 \left[\frac{k^3}{3} + np(1 - k)^2\right]$$

$$k = \sqrt{2np + (np)^2} - np = \sqrt{0.194 + 0.0094} - 0.097 = 0.35$$

$$I_t = 12(8.94)^3 \left[\frac{0.35^3}{3} + 0.097(0.65)^2\right] = 474 \text{ in.}^4$$

$$I_a = 0.5(I_g + I_t) = 0.5(1{,}000 + 474) = 737 \text{ in.}^4$$

$$k_E = \frac{160EI}{L^3} = \frac{160(3)10^3(737)}{144(11^3)} = 1{,}850 \text{ kips/ft}$$

$$y_E = \frac{R_m}{k_E} = \frac{40.6}{1{,}850} = 0.0219 \text{ ft}, \quad y_m = \alpha\ \beta\ y_E = 6(0.0223)$$

$$= 0.1314 \text{ ft (par. 6-26)}$$

$$\text{Weight} = \frac{10(11)(150)}{12(1{,}000)} = 1.375 \text{ kips}$$

$$\text{Mass,} \quad m = \frac{Wt}{g} = \frac{1.375}{32.2} = 0.0427 \frac{\text{kip-sec}^2}{\text{ft}}$$

e. <u>First Trial - Equivalent System Properties.</u>

$$R_{me} = K_L R_m = 0.57(40.6) = 23.1 \text{ kips} \quad \text{(eq 6.12)}$$

$$H_e = K_L H = 0.57(1.87) = 1.07 \text{ kip-sec} \quad \text{(eq 6.8)}$$

$$m_e = K_M m = 0.42(0.0427) = 0.0179 \frac{\text{kip-sec}^2}{\text{ft}} \quad \text{(eq 6.2)}$$

$$W_P = \frac{H_e^2}{2m_e} = \frac{1.07^2}{2(0.0179)} = 32.0 \text{ ft-kips} \quad \text{(eq 6.10)}$$

$$T_n = 2\pi\sqrt{\frac{K_{LM}\ m}{k_E}} = 2\pi\sqrt{\frac{0.78(0.0427)}{1{,}850}} = 0.027 \text{ sec} \quad \text{(eq 6.14)}$$

f. <u>Work Done vs Energy Absorption Capacity.</u>

$$C_T = \frac{T}{T_n} = \frac{0.083}{0.027} = 3.07$$

(eqs 6.15, 6.16)

$$C_R = \frac{R_m}{B} = \frac{4}{45.1} = 0.90$$

$$t_m/T = 0.39 \quad \text{(fig. 5.29)}$$

$$t_m = 0.39(0.083) = 0.032 \text{ sec}$$

$C_W = 0.087$ (fig. 5.27)

$W_m = C_W W_p = 0.087(32.0) = 2.78$ ft-kips (eq 6.17)

$E = R_{me}\left[y_m - 0.5y_E\right] = 23.1\left[0.1314 - 0.5(0.0219)\right]$

$= 2.78$ ft-kips (eq 6.18)

$E = W$ $\therefore$ the selected proportions are satisfactory as a preliminary design.

g. Design of Support Section.

Assume #5 bars

$d = 10 - 1.5 - 0.31 = 8.19$ in.

$$M_p = p\, f_{dy}\, bd^2 \left[1 - \frac{p\, f_{dy}}{1.7f'_{dc}}\right] \quad \text{(eq 4.16)}$$

$$M_p = p(52)(1)(8.19)^2 \left[1 - \frac{p(52)}{1.7(3.9)}\right] = 37.2 \quad \therefore\ p = 0.012$$

$A_s = p\, bd = 0.012(12)(8.19) = 1.18$ in.^{2}

Try #5 bars at 3-1/4 in., $A_s = 1.14$ in.^{2}, $p = \frac{1.14}{12(8.19)} = 0.0116$

$$M_{Ps} = 1.14(52)\,\frac{8.19}{12}\left[1 - \frac{0.0116(52)}{1.7(3.9)}\right] = 36.8 \text{ kip-ft}$$

h. Design of Midspan Section.

$p = 0.0097$, $d = 8.94$

$A_s = p\, bd = 0.0097(12)(8.94) = 1.04$ in.^{2}

Try #5 bars at 3.5 in.

$A_s = \frac{12}{3.5}(0.31) = 1.06$ in.^{2}, $p = 0.0099$

$$M_p = 1.06(52)\,\frac{8.94}{12}\left[1 - \frac{0.0099(52)}{1.7(3.9)}\right] = 37.9 \text{ kip-ft}$$

Run midspan steel through to pinned support.

i. Determination of Maximum Deflection and Dynamic Reactions by Numerical Integration.

$M_{Ps} = 36.8$ kip-ft (par. 9-23g)

$M_{Pm} = 37.2$ kip-ft (par. 9-23d)

$I_a = 737$ in.^{4} (par. 9-23d)

$$\text{Weight} = \frac{10(11)(150)}{12(1{,}000)} = 1.375 \text{ kips}$$

Mass, $m = \frac{1.375}{32.2} = 0.0427 \frac{\text{kip-sec}^2}{\text{ft}}$

Elastic range:

$$R_{1m} = \frac{8M_{Ps}}{L} = \frac{8(36.8)}{11} = 26.8 \text{ kips}$$

$$k_1 = \frac{185EI}{L^3} = \frac{185(3)\,10^3(737)}{144(11^3)} = 2{,}130 \text{ kips/ft}$$

$$y_e = \frac{R_{1m}}{k_1} = \frac{26.8}{2{,}130} = 0.0126 \text{ ft}$$

Elasto-plastic range:

$$R_m = \frac{4}{L}(M_{Ps} + 2M_{Pm}) = \frac{4}{11}[36.8 + 2(37.2)] = 40.9 \text{ kips}$$

$$k_{ep} = \frac{384EI}{5L^3} = \frac{384(3)\,10^3(737)}{5(144)11^3} = 890 \text{ kips/ft}$$

$$y_{ep} = y_e + \frac{R_m - R_{1m}}{k_{ep}} = 0.0132 + \frac{40.9 - 26.8}{890} = 0.0284 \text{ ft}$$

Plastic range:

$$R_m = \frac{4}{L}(M_{Ps} + 2M_{Pm}) = 40.9 \text{ kips}$$

$$y_m = \alpha\beta\, y_E = \alpha\beta \frac{R_m}{k_E} = 6\,\frac{40.9}{1{,}850} = 0.1326 \text{ ft}$$

The basic equation for the numerical integration in table 9.29 is

$$y_{n+1} = 2\,y_n - y_{n-1} + \ddot{y}_n \,\Delta t^2 \quad \text{(eq 5.49)}$$

where

$$\ddot{y}_n = \frac{P_n - R_n}{K_{LM}(m)}$$

The time interval $\Delta t = 0.0025$ sec is approximately $T_n/10$ (par. 5-08)

$$\ddot{y}_n\,\Delta t^2 = \frac{(0.0025)^2}{0.78(0.0427)}(P_n - R_n) = 1.8765(10^{-4})(P_n - R_n),$$

elastic range

$$\ddot{y}_n\,\Delta t^2 = \frac{(0.0025)^2}{0.78(0.0427)}(P_n - R_n) = 1.8765(10^{-4})(P_n - R_n),$$

elasto-plastic range

$$\ddot{y}_n\,\Delta t^2 = \frac{(0.0025)^2}{0.66(0.0427)}(P_n - R_n) = 2.2177(10^{-4})(P_n - R_n),$$

plastic range

Table 9.29. Determination of Maximum Deflection and Dynamic Reactions for Front Wall Slab

t (sec)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n (\Delta t)^2$ (ft)	y_n (ft)	V_{1n} (kips)	V_{2n} (kips)
0	45.1	0	22.5	0.00422	0	5.4	8.6
0.0025	43.7	9.0	34.7	0.00651	0.00422	7.6	12.2
0.005	42.4	28.9	13.5	0.00253	0.01495	15.9	15.9
0.0075	41.0	40.7	0.3	0.00006	0.02821	20.4	20.4
0.010	39.7	40.9	-1.2	-0.00027	0.04153	20.3	20.3
0.0125	38.3	40.9	-2.6	-0.00058	0.05458	20.1	20.1
0.015	36.9	40.9	-4.0	-0.00089	0.06705	20.0	20.0
0.0175	35.6	40.9	-5.3	-0.00117	0.07863	19.8	19.8
0.020	34.2	40.9	-6.7	-0.00149	0.08904	19.6	19.6
0.0225	32.9	40.9	-8.0	-0.00177	0.09796	19.5	19.5
0.025	31.5	40.9	-9.4	-0.00208	0.10511	19.3	19.3
0.0275	30.2	40.9	-10.7	-0.00237	0.11018	19.2	19.2
0.030	28.8	40.9	-12.1	-0.00268	0.11288	19.0	19.0
0.0325	27.4	40.9	-13.5	-0.00299	0.11290	18.8	18.8
0.035	26.1	34.6	-8.5	-0.00160	0.10993	12.1	19.8
0.0375	24.7	24.8			0.10536	9.4	15.4
0.040	23.4					8.9	14.5
0.0425	22.2					8.5	13.7
0.045	20.9					7.9	13.0
0.0475	19.5					7.4	12.1
0.050	18.1					6.9	11.2
0.0525	16.8					6.4	10.4
0.055	15.5					5.9	9.6
0.0575	15.4					5.9	9.5
0.060	15.3					5.8	9.5
0.0625	15.1					5.8	9.4
0.065	15.0					5.7	9.3
0.0675	14.9					5.7	9.2
0.070	14.8					5.6	9.2
0.0725	14.7					5.6	9.1
0.075	14.5					5.5	9.0
0.0775	14.4					5.5	8.9
0.080	14.3					5.4	8.9
0.0825	14.1					5.4	8.7
0.085	13.9					5.3	8.6
0.0875	13.2					5.0	8.2
0.090	13.4					5.1	8.3
0.0925	13.2					5.0	8.2
0.095	13.1					5.0	8.1

The dynamic reaction equations are listed in paragraph 9-23c. The P_n values for the second column are obtained from figure 9.30 multiplying by 144(11)/1,000 = 1.584.

At time t = 0.0375 sec, in table 9.29, the resistance R_n becomes equal to the applied load. At times greater than this, R_n will oscillate about the applied load. However, the resistance values are assumed equal to the load for computing the reactions for times greater than t = 0.0375 as this procedure is conservative, and avoids the necessity for continuing the dynamic analysis.

j. Shear Strength and Bond Stress at Fixed End.

$V_{max} = 20.4$ kips (table 9.29)

Allowable $v_y = 0.04\, f_c' + 5{,}000\, p + rf_y$ (eq 4.24)

For no shear reinforcement $v_y = 0.04(3{,}000) + 5{,}000(0.0116)$

$= 120 + 58 = 178$ psi

$$v = \frac{8V}{7bd} = \frac{8(20{,}400)}{7(12)(8.19)} = 237 \text{ psi}$$

Shear reinforcement required for 237 - 178 = 59 psi

$$r = \frac{59}{40{,}000} = 0.0015$$

Try 1 #3, $A_s = 0.11$

$$r = A_s/bs = \frac{0.11}{12\ s} = 0.0015, \quad \therefore \quad s = 6.1 \text{ in.}$$

Use 1 #3 per foot of width at 6 in.

$$u = \frac{8V}{7\ \Sigma o\ d} = \frac{8(20{,}400)}{7(7.25)8.19} = 393 \text{ psi} \quad \text{OK}$$

Allowable $u = 0.15\ f_c' = 0.15(3{,}000) = 450$ psi

k. Shear Strength and Bond Stress at Pinned End.

$V_{max} = 20.4$ kips (table 9.29)

Allowable $v_y = 0.04\ f_c' + 5{,}000\ p + rf_y$

For no shear reinforcement $v_y = 0.04(3{,}000) + 5{,}000(0.0099)$

$= 120 + 50 = 170$ psi

$$v = \frac{8V}{7bd} = \frac{8(20{,}400)}{7(12)(8.94)} = 217 \text{ psi}$$

Shear reinforcement required for 217 - 170 = 47 psi

$$r = \frac{47}{40,000} = 0.0012$$

Try 1 #3, $A_s = 0.11$

$$r = A_s/bs = \frac{0.11}{12\ s} = 0.0012 \quad \therefore \ s = 7.6 \text{ in.}$$

Use 1 #3 per foot at 6 in.

$$u = \frac{8V}{7\,\Sigma o\ d} = \frac{8(20,400)}{7(6.73)8.94} = 387 \text{ psi} \quad \text{OK}$$

Allowable $u = 0.15\ f'_c = 0.15(3,000) = 450$ psi

l. Back Wall Slab Analysis. Using the same resistance functions as for the front wall slab a computation of back wall reactions is made by means of a numerical analysis below. These reactions are necessary in order to compute the net external forces acting in the overturning and sliding analysis presented in paragraph 9-24. $\overline{P}_{back}$ is obtained from figure 9.31.

m. Reaction Determination by Numerical Integration.

Table 9.30. Determination of Maximum Deflection and Dynamic Reactions for Back Wall Slab

t (sec)	$\overline{P}_{back}$ (psi)	P_n (kips)	R_n (kips)	$P_n - R_n$ (kips)	$\ddot{y}_n(\Delta t)^2$ (ft)	y (ft)	Strain Range	V_1 (kips)	V_2 (kips)
0.035	0.1	0.174	0	0.174	0.000020	0	e	0.02	0.02
0.0375	0.25	0.396	0.043	0.353	0.000070	0.00002	e	0.05	0.04
0.040	0.40	0.634	0.234	0.400	0.000075	0.00011	e	0.13	0.15
0.0425	0.55	0.872	0.586	0.286	0.000054	0.000275	e	0.26	0.33
0.045	0.70	1.110	1.040	0.070	----	0.00049	e	0.40	0.55
0.0475	0.85	1.350	1.490	-0.140	-0.000020	0.00070	e	0.54	0.80
0.050	1.05	1.600	1.90	-0.300	-0.000056	0.00089	e	0.68	1.01
0.0525	1.20	1.900	2.18	-0.28	-0.000053	0.001024	e	0.80	1.20
0.055	1.35	2.140	2.34	-0.20	-0.000037	0.001105	e	0.87	1.42
0.0575	1.55	2.460	2.44	+0.02	----	0.001149	e	0.93	1.52
0.060	1.70	2.70	2.54	+0.16	+0.000030	0.001193	e	0.99	1.61
0.0625	1.90	3.01	2.70	+0.31	+0.000058	0.001267	e	1.06	1.73
0.065	2.02	3.20	2.98	+0.22	+0.000041	0.001399	e	1.16	1.89
0.0675	2.25	3.57	3.35	+0.22	+0.000041	0.001572	e	1.30	2.12
0.070	2.40	3.80	3.80	----	----	0.001786	e	1.45	2.35
0.0725	2.55	4.05	4.26	-0.21	-0.000040	0.002000	e	1.60	2.60
0.075	2.75	4.36	4.63	-0.27	-0.000051	0.002174	e	1.74	2.82
0.0775	2.90	4.60	4.88	-0.22	-0.000041	0.002297	e	1.82	2.97
0.080	3.10	4.92	5.06	-0.14	-0.000026	0.002379	e	1.90	3.12
0.0825	3.23	5.11	5.20	-0.09	-0.000017	0.002435	e	1.96	3.21
0.085	3.25	5.15	5.26	-0.11	-0.000021	0.002474	e	1.98	3.24
0.0875	3.66	5.81	5.32	+0.49	+0.000092	0.002492	e	2.07	3.39
0.090	3.75	5.95	5.55	+0.40	+0.000075	0.002602	e	2.15	3.51
0.0925	3.88	6.16	5.95	+0.21	+0.000040	0.002787	e	2.28	3.73
0.095	4.0	6.35	6.40	-0.05	-0.000009	0.003012	e	2.42	3.96

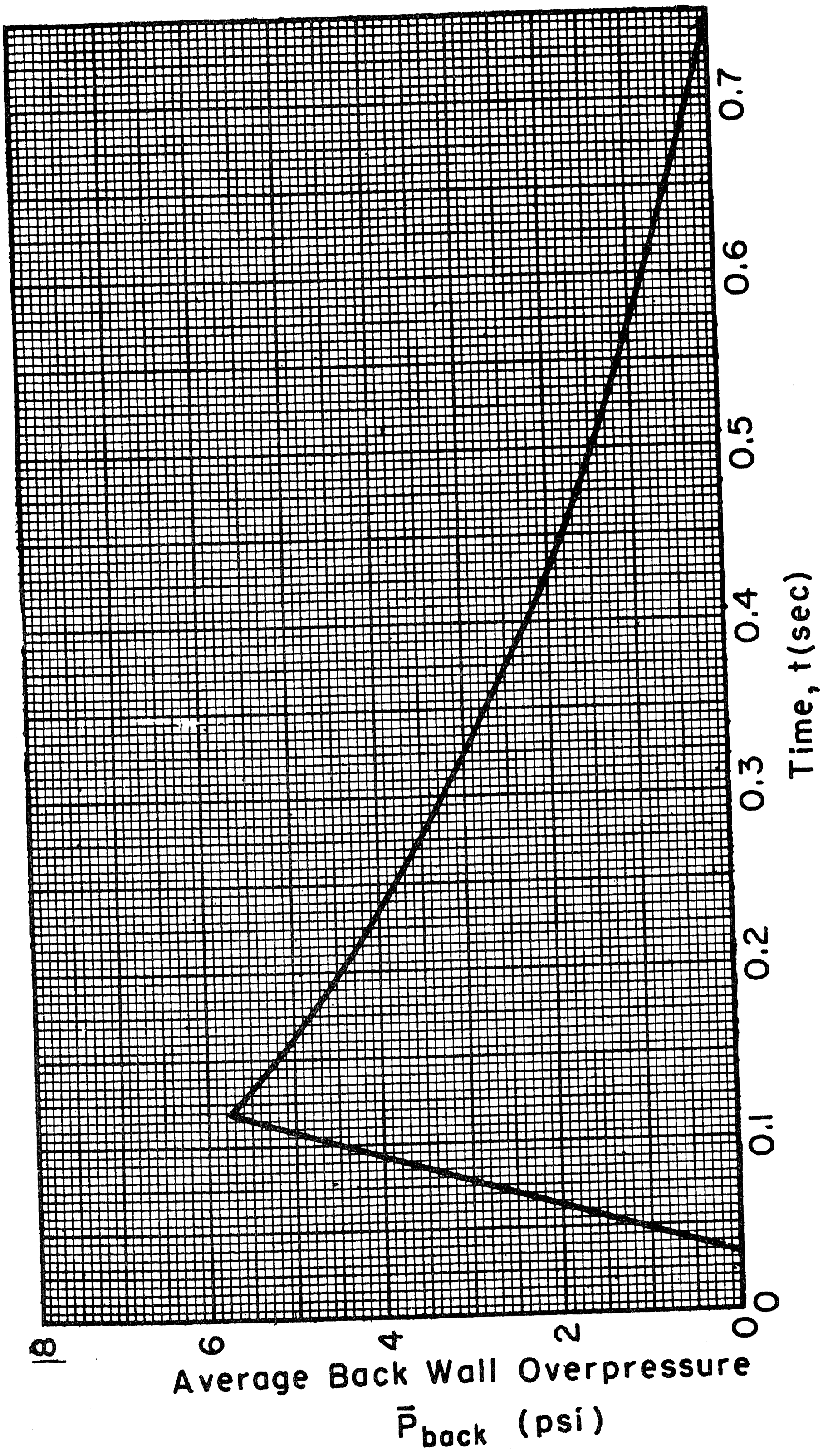


Figure 9.31. Average back wall overpressure vs time curve, P_{back} (see par. 3-08b)

9-24 RIGID BODY OVERTURNING AND SLIDING ANALYSIS. In order to perform a reasonably accurate preliminary shear wall analysis it is necessary to initially analyze the building as a rigid body, sliding and rotating about the longitudinal axis of the building. Using the principles presented in paragraph 9-06a for the rigid body analysis, the horizontal acceleration $\ddot{x}_o$ and the angular acceleration α_o about the longitudinal axis through the bottom of the footings are determined. The corresponding horizontal displacement x_o and angular displacement θ are determined as a function of time by means of a concurrent numerical integration of equations (9.4) and (9.5). From this analysis one may then obtain the rigid body accelerations of the walls, floors, and roof slab which are used to determine the rigid body inertial forces acting upon the shear walls in the preliminary shear wall analysis.

$$\alpha_o = \frac{M_o - F_o\bar{y}}{I_o - m\bar{y}^2} \qquad \text{(eq 9.4)}$$

$$\ddot{x}_o = \frac{F_o}{m} - \alpha_o\bar{y} \qquad \text{(eq 9.5)}$$

a. Mass Moment of Inertia (I_o). This consists of the mass moment of inertia of all of the elements about the axis of rotation of the structure. Because of the tendency for certain parts of the structure such as the earth, floor slab, column footings, etc., to translate but rotate, these elements will have an inertial component in the horizontal direction only. All other portions of the structure will have inertial components in both the vertical and horizontal directions. The centroid and moment of inertia computations are presented in tables 9.31, 9.32, and 9.33. The dimensions

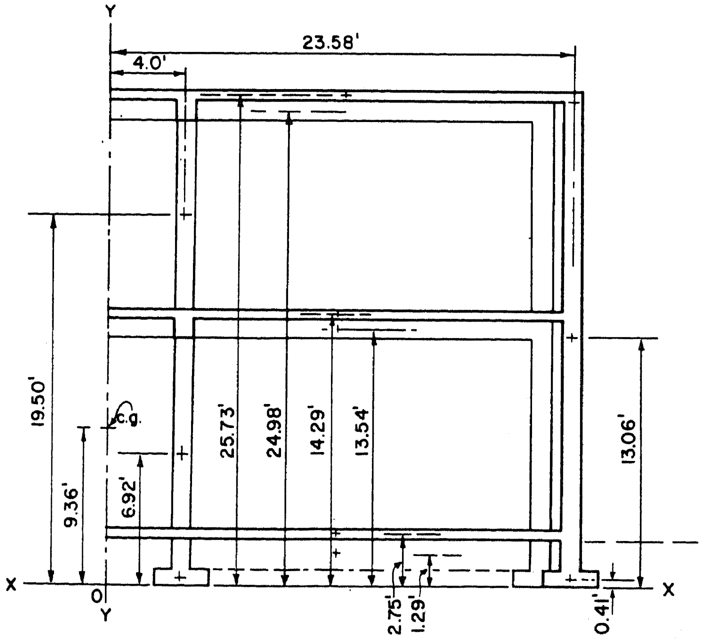


Table 9.31. Computation of Mass and Location of Centroid of Building

Element	Dimensions	Volume (cu ft)	Weight (kips)	Mass $\left(\frac{\text{kip-sec}^2}{\text{ft}}\right)$	$\bar{y}$ (ft)	$m\bar{y}$
Front wall	(0.83)24.64(156.8)	3,220	483	15.05	13.06	196.2
Rear wall	(0.83)24.64(156.8)	3,220	483	15.05	13.06	196.2
End walls	2(0.83)[(24.6)46.34-6.0(7.5)]	1,825	274	8.50	13.06	111.7
Shear walls	4(0.83)[(24.6)46.34-14.0(7.5)]	3,450	517	16.05	13.06	210.9
Roof slab	(0.54)48.0(156.8)	4,060	608	18.90	25.73	486.0
Trans roof girders	16(1.0)1.33(19.0)	406	60.8	1.89	24.98	47.2
Long. roof girders	2(1.0)1.0(155.2)	310	46.5	1.44	24.98	36.0
Second floor columns	36.0(1.0)1.0(10.0)	360	54.0	1.68	19.50	32.8
Second floor slab	0.42(46.3)155.2	3,018	453	14.07	14.29	201.1
Second floor trans girders	16(1.0)1.0(19.0)	304	45.6	1.42	13.54	19.2
Second floor long. girders	2(1.0)1.0(155.2)	311	46.8	1.46	13.54	19.8
First floor slab	0.50(46.3)155.2	3,600	540	16.80	2.75	46.2
First floor columns	36(1.0)1.0(12.0)	432	64.6	2.01	6.92	13.9
Earth	[1.7(46.3)155.2] + [0.83(44.5)153]	17,920	1,792	55.70	1.27	71.7
Isolated footings	16(3.5)3.5(1.17)	229	34.4	1.07	0.58	0.6
Attached footings	20(1.67)2.67(0.83)	74	11.1	0.34	0.41	0.1
Front and rear footings	2(2.5)0.83(158.5)	662	99.3	3.08	0.41	1.3
Shear and end footings	6(2.5)2.5(44.5)	1,670	250	7.75	1.25	9.7
Occupancy load (second floor)	0.015(46.3)155.2		108	3.35	15.5	51.9
Occupancy load (first floor)	0.015(46.3)155.2		108	3.35	4.0	13.4
			6,079.1	188.96		1,768.9

$$\bar{y} = \frac{1,768.9}{188.96} = 9.36 \text{ ft}$$

Table 9.32. Computation of Mass of Rotating Elements Only

Element	Ratio Rotating		Entire Mass	Rotating Mass
Roof slab	$1 - \frac{2[28(12) + 1.5(28)24]}{48(157)}$			
	1 - 0.358 = 0.642	0.642	18.90	12.12
Transverse roof girders		0.50	1.89	0.95
Longitudinal roof girders	60/155	0.39	1.44	0.56
Second floor columns	20/36	0.56	1.68	0.94
Second floor slab	(Same ratio as roof slab above)	0.642	15.40	9.90
Second floor transverse girders	(Same ratio as roof girders above)	0.50	1.42	0.71
Second floor longitudinal girders	(Same ratio as roof girders above)	0.39	1.46	0.57
First floor slab	(Same ratio as roof slab above)	0.642	16.80	10.80
First floor columns	(Same ratio as second floor columns)	0.56	2.01	1.12
Occupancy load	(Same ratio as roof slab)	0.642	3.35	2.15

Table 9.33. Computation of Polar Moment of Inertia, I_o

Element	$1/12\ md^2$	$m\bar{x}^2$	I_{YY} (kip-sec²/ft)
Front wall	$(1/12)15.05(0.83)^2 = 0.87$	$15.05(23.58)^2 = 8350$	8,351
Rear wall	$(1/12)15.05(0.83)^2 = 0.87$	$15.05(23.58)^2 = 8350$	8,351
End walls	$(1/12)8.50(46.34)^2 = 1,525$		1,525
Shear walls	$(1/12)16.05(46.34)^2 = 2,870$		2,870
Roof slab	$(1/12)[18.90(48.0)^2 - 6.78(28.0)^2] =$		3,180
Transverse roof girders	$(1/12)0.95(19.0)^2 = 28.6$	$0.95(13.0)^2 = 161$	190
Longitudinal roof girders	$(1/12)0.56(1.0)^2 = 0.05$	$0.56(4.0)^2 = 9$	9
Second floor column	$(1/12)0.94(1.0)^2 = 0.07$	$0.94(22.0)^2 = 455$	455
Second floor slab	$(1/12)[15.40(46.3)^2 - 5.50(28.0)^2] =$		2,400
Second floor transverse girders	$(1/12)0.71(19.0)^2 = 21.40$	$0.71(13.0)^2 = 120$	141
Second floor longitudinal girders	$(1/12)0.56(1.0)^2 = 0.05$	$0.56(4.0)^2 = 9$	9
First floor slab	$(1/12)[16.8(46.3)^2 - 6.0(28.0)^2] =$		2,610
First floor columns	$(1/12)1.12(1.0)^2 = 0.09$	$1.12(22.0)^2 = 542$	542
Earth			-----
Isolated footings			-----
Attached footings	$(1/12)0.34(1.67)^2 = 0.08$	$0.34(22.0)^2 = 165$	165
Front and rear footings	$(1/12)3.08(2.5)^2 = 1.60$	$3.08(23.5)^2 = 1,700$	1,702
Shear and end footings	$(1/12)7.75(44.5)^2 = 1,280$		1,280
Occupancy load	$2(1/12)[3.35(46.3)^2 - 1.20(28.0)^2]c =$		1,035
			34,667

Element	$1/12\ md^2$	$m\bar{y}^2$	I_{XX} (kip-sec²/ft)
Front wall	$(1/12)15.05(24.64)^2 = 762$	$15.05(13.06)^2 = 2,560$	3,322
Rear wall	$(1/12)15.05(24.64)^2 = 762$	$15.05(13.06)^2 = 2,560$	3,322
End walls	$(1/12)8.50(24.64)^2 = 432$	$8.50(13.06)^2 = 1,450$	1,882
Shear walls	$(1/12)16.05(24.64)^2 = 815$	$16.05(13.06)^2 = 2,740$	3,555
Roof slab	$(1/12)18.90(0.54)^2 = 0.46$	$18.90(25.73)^2 = 12,550$	12,550
Transverse roof girders	$(1/12)1.89(1.0)^2 = 0.16$	$1.89(24.86)^2 = 1,172$	1,172
Longitudinal roof girders	$(1/12)1.44(1.0)^2 = 0.12$	$1.44(24.86)^2 = 892$	892
Second floor columns	$(1/12)1.68(10.0)^2 = 14.0$	$1.68(19.50)^2 = 638$	652
Second floor slab	$(1/12)15.40(0.46)^2 = 0.27$	$15.40(14.27)^2 = 3,140$	3,140
Second floor transverse girders	$(1/12)1.42(1.0)^2 = 0.11$	$1.42(13.36)^2 = 254$	254
Second floor longitudinal girders	$(1/12)1.46(1.0)^2 = 0.12$	$1.46(13.36)^2 = 261$	261
First floor slab	$(1/12)16.80(0.50)^2 = 0.35$	$16.80(2.57)^2 = 127$	127
First floor columns	$(1/12)2.01(12.0)^2 = 24.1$	$2.01(7.0)^2 = 99$	123
Earth	$(1/12)55.70(2.50)^2 = 28.8$	$55.70(1.29)^2 = 92.5$	121
Isolated footings	$(1/12)1.07(1.17)^2 = 0.12$	$1.07(0.58)^2 = 0.3$	-----
Attached footings	$(1/12)0.34(0.83)^2$ ---	$0.34(0.41)^2 = 0.05$	-----
Front and rear footings	$(1/12)3.08(0.83)^2 = 0.17$	$3.08(0.41)^2 = 0.5$	1
Shear and end footings	$(1/12)7.75(2.5)^2 = 4.04$	$7.75(1.25)^2 = 12.1$	16
Occupancy load (second)		$3.35(15.5)^2 = 804$	804
Occupancy load (first)		$3.35(4.0)^2 = 53.6$	54
			32,248

$$I_o = I_{YY} + I_{XX} = 34,667 + 32,248 = 66,915 \text{ kip-sec}^2/\text{ft}$$

used in these computations are shown in the sketch on page 124.

b. Ground Foundation Interaction. The rotation of the structure under the blast loads develops a resisting moment of the vertical soil pressures against the footings. The magnitude of this moment reaction which tends to resist the overturning of the structure is given by:

$$B = \frac{\pi EL^2}{4(1 - \nu)^2} \quad \text{(eq 4.59)}$$

To apply this formula to a specific structure the ratio I_{net}/I_{gross} must be determined, where I_{net} is the moment of inertia of the actual footing area and I_{gross} is the moment of inertia of a solid rectangular area extending over the entire extent of the footings, both quantities being evaluated about the longitudinal axis about which the structure is rotating. The net overturning resistance of the soil is given by the product of equation (4.57) and the ratio I_{net}/I_{gross}.

$$\frac{I_{net}}{I_{gross}} = \frac{(1/12)bd^3 - (1/12)b'd'^3}{(1/12)bd^3} = 1 - \frac{b'd'^3}{bd^3}$$

$$b = 156.8 + 1.6 = 158.4 \text{ ft}$$

$$d = 48.0 + 1.6 = 49.6 \text{ ft} = 2L$$

$$b' = 158.4 - 6(2.5) = 143.4 \text{ ft}$$

$$d' = 49.6 - 2(2.50) = 44.6 \text{ ft}$$

$$\frac{I_{net}}{I_{gross}} = 1 - \frac{143.4(44.6)^3}{158.4(49.6)^3} = 0.342$$

$$B_{gross} = \frac{\pi(40)}{4(1 - 0.333^2)} \left[\frac{49.6(12)^2}{2}\right] = 31.2(10)^5 \text{ ft-kips/ft/radian}$$

$$B_{net} = 31.2(10)^5\ 0.342(158.5) = 17.0(10)^7 \text{ ft-kips/radian}$$

c. Passive Resistance of Soil. Since the earth between the front and back walls is considered to move with the structure, the back wall only will develop horizontal passive soil resistance against translation of the structure.

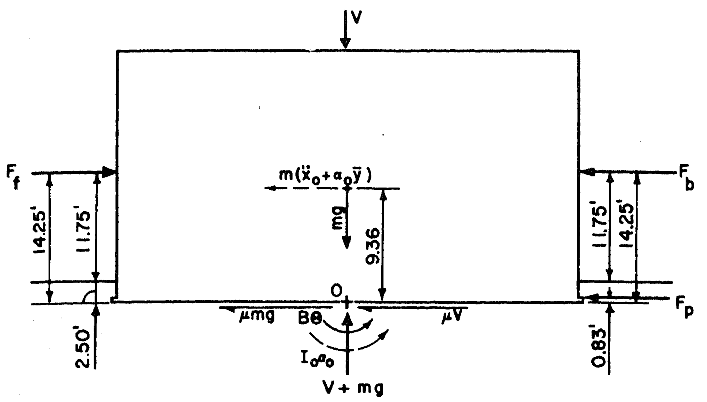
$$F_p = 0.5(\gamma)(H^2)(K_{P\phi}) \quad \text{(eq 4.58)} \quad = (0.5)0.100(2.5)^2\ 10$$

$$= 3.12 \text{ kips/ft} = (3.12)157.0 = 490 \text{ kips} = \text{total passive resistance}$$

d. <u>Determination of Total Blast Forces on Front Wall F_f, Back Wall F_b, and Roof V.</u> The total air blast loads acting upon the front wall, back walls, and roof of the structure, respectively, are used in the rigid body overturning and sliding analysis. These values are determined from the product of the area of these surfaces and the air blast pressures acting upon them as a function of time. These values are tabulated in table 9.34.

Table 9.34. $(F_f - F_b)$ and V Values for Rigid Body Analysis

t (sec)	$\overline{P}_{front} - \overline{P}_{back}$ (psi)	$F_f - F_b$ (kips)	$\overline{P}_{front} + \overline{P}_{back}$ (psi)	V_f (kips)	$\overline{P}_{roof}$ (psi)	V_r (psi)	V (kips)	μV (kips)
0	28.5	15,150	28.5	487	0	0	487	365
0.0025	27.7	14,700	27.7	473	0.63	684	1,157	867
0.005	26.8	14,220	26.8	458	1.26	1,368	1,826	1,370
0.0075	25.9	13,750	25.9	443	1.89	2,050	2,493	1,870
0.010	25.1	13,310	25.1	428	2.52	2,730	3,158	2,360
0.0125	24.2	12,850	24.2	414	3.15	3,420	3,834	2,870
0.015	23.3	12,350	23.3	398	3.78	4,100	4,498	3,360
0.0175	22.5	11,950	22.5	385	4.41	4,780	5,165	3,870
0.020	21.6	11,450	21.6	369	5.04	5,460	5,829	4,370
0.0225	20.8	11,050	20.8	356	5.67	6,150	6,506	4,880
0.025	19.9	10,570	19.9	340	6.30	6,840	7,180	5,380
0.0275	19.1	10,150	19.1	326	6.93	7,520	7,846	5,870
0.030	18.2	9,660	18.2	311	7.56	8,200	8,511	6,380
0.0325	17.4	9,240	17.4	298	8.20	8,900	9,198	6,900
0.035	16.6	8,820	16.6	284	8.65	9,370	9,654	7,240
0.0375	15.5	8,240	16.0	274	8.50	9,220	9,490	7,100
0.040	14.4	7,650	15.2	260	8.40	9,100	9,360	7,020
0.0425	13.4	7,120	14.6	250	8.30	9,000	9,250	6,940
0.045	12.4	6,580	14.0	239	8.20	8,900	9,139	6,850
0.0475	11.4	6,060	13.2	226	8.12	8,800	9,026	6,770
0.050	10.3	5,460	12.5	214	8.05	8,730	8,944	6,700
0.0525	9.4	5,000	11.8	202	8.00	8,680	8,882	6,660
0.055	8.5	4,520	11.1	190	7.95	8,620	8,810	6,620
0.0575	8.1	4,300	11.3	193	7.85	8,500	8,693	6,520
0.060	7.9	4,190	11.6	198	7.75	8,380	8,578	6,440
0.0625	7.60	4,040	11.4	195	7.65	8,280	8,475	6,350
0.065	7.45	3,960	11.5	196	7.55	8,170	8,366	6,260
0.0675	7.15	3,800	11.65	199	7.45	8,060	8,259	6,200
0.070	6.9	3,670	11.7	200	7.35	7,950	8,150	6,110
0.0725	6.7	3,560	11.8	202	7.25	7,840	8,042	6,040
0.075	6.43	3,420	11.93	204	7.15	7,740	7,944	5,950
0.0775	6.20	3,300	12.00	205	7.07	7,650	7,855	5,880
0.080	5.9	3,140	12.1	207	7.00	7,580	7,787	5,830
0.0825	5.7	3,030	12.15	208	6.90	7,460	7,668	5,750
0.085	5.57	2,960	12.2	209	6.80	7,360	7,569	5,680
0.0875	5.3	2,820	12.3	210	6.70	7,250	7,460	5,600
0.090	4.95	2,630	12.45	213	6.60	7,140	7,353	5,510
0.0925	4.70	2,500	12.5	214	6.55	7,090	7,304	5,480
0.095	4.30	2,290	12.9	220	6.45	6,980	7,200	5,400



The net lateral force is the value desired for the rigid body analysis and hence is contained in the tabulation. The vertical blast load on the roof and footings is multiplied by the coefficient of friction μ to give the lateral friction resistance to translation of the structure.

$$F_f = (23.5)156.8\left(\frac{144}{1{,}000}\right)\overline{P}_{front} = 532\ \overline{P}_{front} \text{ kips}$$

$$F_b = 532\ \overline{P}_{back} \text{ kips}$$

V = vertical blast load on roof and portion of footings exposed to blast

$$\text{Blast load on roof} = (48.0)156.8\left(\frac{144}{1{,}000}\right)\overline{P}_{roof} = 17.1\ \overline{P}_{roof}$$

The average net lateral overpressure vs time curve (fig. 9.32) is obtained by subtracting $\overline{P}_{back}$ values (fig. 9.31) from $\overline{P}_{front}$ values (fig. 9.30).

e. Dynamic Analysis. The dynamic rigid body sliding and overturning analysis is performed through the tabulations contained in table 9.35. A simultaneous numerical integration is performed upon the quantities α_o and $\ddot{x}_o$ to obtain the time history of the rigid body rotation θ and the rigid body translation x_o. The acceleration impulse extrapolation method (eq 5.49 of par. 5-08d) is used for the dynamic analysis as follows:

$$(\theta)_{t=n+1} = 2(\theta)_{t=n} - (\theta)_{t=n-1} + (\alpha_o)_{t=n}(\Delta t)^2$$

$$(x_o)_{t=n+1} = 2(x_o)_{t=n} - (x_o)_{t=n-1} + (\ddot{x}_o)_{t=n}(\Delta t)^2$$

Use $\Delta t = 0.0025$ sec (Same time interval used for roof and front wall.)

$$\ddot{x}_o = \frac{F_o}{m} - \alpha_o\,\bar{y} \quad \text{(eq 9.5)}$$

While the structure is sliding:

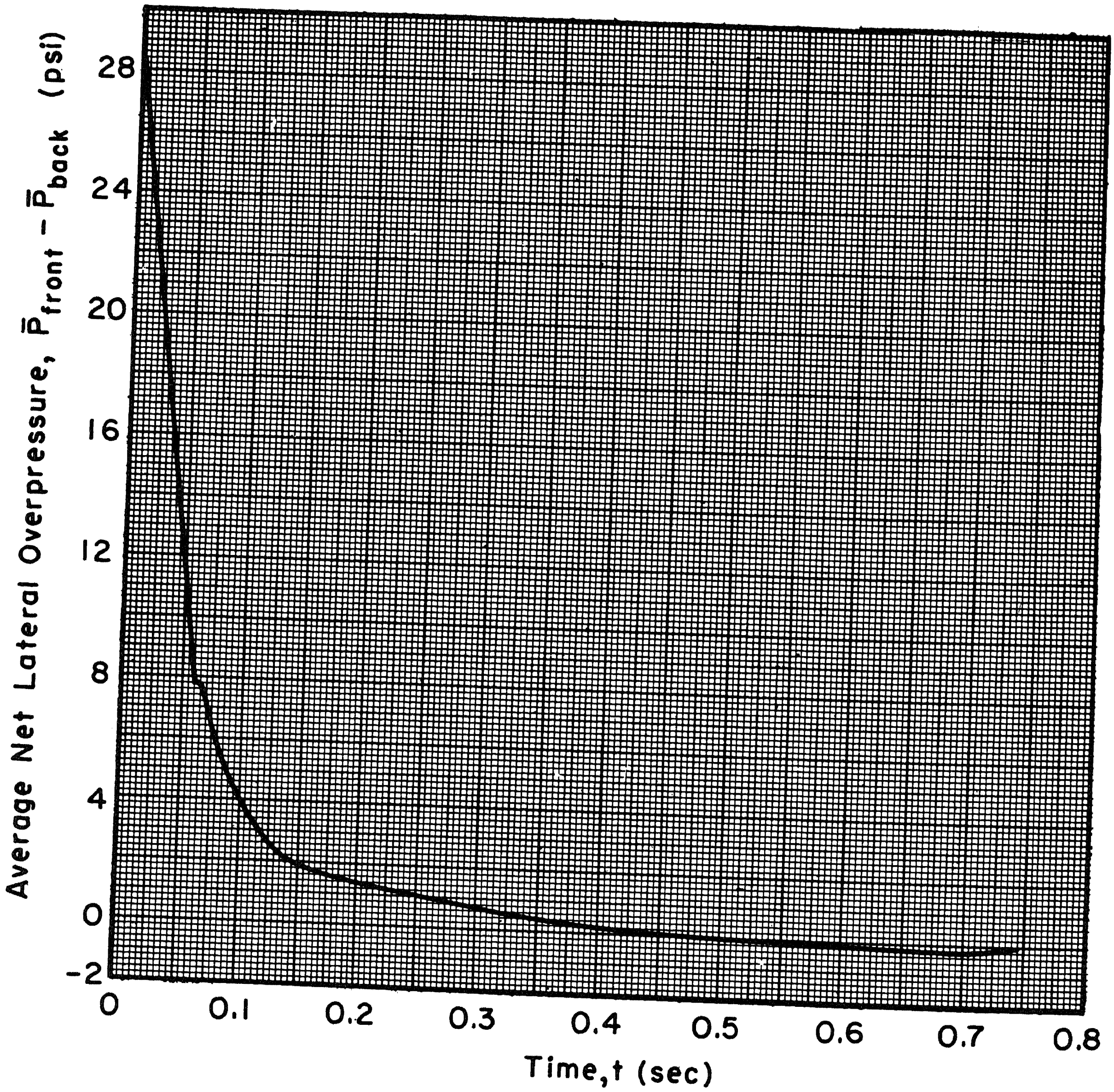


Figure 9.32. Average net lateral overpressure vs time curve

$$\alpha_o = \frac{M_o - F_o \bar{y}}{I_o - m\bar{y}^2} \quad \text{(eq 9.4)}$$

where

M_o = moment of all external forces about axis of rotation "O"

F_o = summation of all external horizontal forces applied to the structure

For this example

Table 9.35. Simultaneous Rigid Body Overturning and Sliding Analysis

t (sec)	$F_f - F_b$ (kips)	$F_f y_f - F_b y_b$ (ft-kips)	$F_p y_p$ (ft-kips)	$B\theta$ (ft-kips)	M_o (ft-kips)	μV (kips)	μmg (kips)	F_o (kips)	$M_o - F_o \bar{y}$ (ft-kips)	α_o (radians/sec^2)	$\alpha_o(\Delta t)^2$ (10^{-5} radians)	θ (10^{-5} radians)	$\frac{F_o}{m_o}$ (ft/sec^2)	$\alpha_o \bar{y}$ (ft/sec^2)	$\ddot{x}_o$ (ft/sec)^2	$\ddot{x}_o(\Delta t)^2$ (10^{-5} ft)	x_o (10^{-5} ft)
0	15,150	216,000	400	0	215,600	365	4,570	9,725	125,600	2.50/2	0.78	0	51.6	11.7	39.9/2	12.5	0
0.0025	14,700	210,000	400	1,325	208,275	867	4,570	8,773	126,075	2.52	1.58	0.78	46.4	23.6	22.8	14.3	12.5
0.005	14,220	203,000	400	5,340	197,260	1,370	4,570	7,790	124,260	2.47	1.55	3.14	41.3	23.1	18.2	11.4	39.3
0.0075	13,750	196,000	400	12,000	183,600	1,870	4,570	6,820	119,900	2.38	1.49	7.05	36.1	22.3	13.8	8.64	77.5
0.010	13,310	190,000	400	21,200	168,400	2,360	4,570	6,190	110,400	2.19	1.37	12.45	32.8	20.5	12.5	7.8	124.3
0.0125	12,850	183,000	400	32,700	149,900	2,870	4,570	4,920	103,700	2.06	1.29	19.22	26.1	19.3	6.8	4.3	178.9
0.015	12,350	176,000	400	46,300	129,300	3,360	4,570	3,930	92,500	1.84	1.15	27.28	20.8	17.2	3.6	2.3	237.8
0.0175	11,950	170,000	400	62,000	107,600	3,870	4,570	3,020	79,400	1.58	0.99	36.49	16.0	14.8	1.2	0.75	299.0
0.020	11,450	163,000	400	79,200	83,400	4,370	4,570	2,020	64,500	1.28	0.80	46.69	10.7	12.0	-1.3	-0.81	360.9
0.0225	11,050	157,500	400	98,000	59,100	4,880	4,570	1,110	48,700	0.97	0.61	57.69	5.9	9.1	-3.2	-2.0	422.0
0.025	10,570	151,000	400	118,000	32,600	5,380	4,570	130	31,380	0.62	0.39	69.30	0.69	5.82	-5.13	-3.21	481.1
0.0275	10,150	144,500	400	138,000	6,100	5,870	4,570	-780	13,400	0.27	0.17	81.30	-4.13	2.53	-6.66	-4.16	537.0
0.030	9,660	138,000	400	176,000	-38,400	6,380	4,570	-1,780	-36,700	-0.73	-0.46	103.47	-9.4	-6.85	-2.55	-1.59	588.7
0.0325	9,240	131,800	400	213,000	-81,600	6,900	4,570	-2,720	-56,100	-1.12	-0.80	125.18	-14.4	-7.50	-6.9	-4.32	638.8
0.035	8,820	126,000	400	248,000	-122,400	7,240	4,570	-3,480	-89,800	-1.78	-1.12	146.09	-18.4	-10.50	-7.9	-4.94	684.6
0.0375	8,240	117,500	400	282,000	-164,900	7,100	4,570	-3,920	-128,100	-2.55	-1.59	165.88	-20.8	-23.9	+3.1	+1.94	725.5
0.040	7,650	109,000	400	313,000	-204,400	7,020	4,570	-4,430	-162,800	-3.24	-2.02	184.08	-23.5	-30.3	+6.8	+4.25	768.3
0.0425	7,120	101,500	400	338,000	-236,900	6,940	4,570	-4,480	-194,900	-3.88	-2.42	198.26	-23.7	-36.3	+12.6	+7.87	815.4
0.045	6,580	94,000	400	357,000	-263,400	6,850	4,570	-5,330	-213,400	-4.24	-2.67	210.02	-28.3	-39.8	+11.5	+7.18	870.4
0.0475	6,060	86,500	400	372,000	-285,900	6,770	4,570	-5,770	-231,900	-4.62	-2.88	219.11	-30.5	-43.20	+12.7	+7.95	932.6
0.050	5,460	77,800	400	384,000	-306,600	6,700	4,570	-6,210	-248,400	-4.92	-3.08	225.32	-32.9	-46.20	+13.3	+8.32	1,002.7
0.0525	5,000	71,400	400	388,000	-317,000	6,660	4,570	-6,720	-254,200	-5.06	-3.16	228.45	-35.6	-47.4	+11.8	+7.37	1,081.1
0.055	4,520	64,500	400	388,000	-323,900	6,620	4,570	-7,160	-256,900	-5.11	-3.20	228.42	-37.9	-47.9	+10.0	+6.25	1,166.9
0.0575	4,300	61,200	400	383,000	-322,200	6,520	4,570	-7,280	-254,000	-5.04	-3.15	225.19	-38.6	-47.3	+8.7	+5.45	1,259.0
0.060	4,190	59,700	400	372,000	-312,700	6,440	4,570	-7,310	-244,200	-4.86	-3.04	218.81	-38.7	-45.5	+6.8	+4.25	1,357.0
0.0625	4,040	57,800	400	355,000	-297,600	6,350	4,570	-7,370	-229,300	-4.55	-2.84	209.39	-38.6	-42.6	+4.0	+2.5	1,459.0
0.065	3,960	56,600	400	335,000	-278,800	6,260	4,570	-6,870	-214,600	-4.26	-2.66	197.13	-36.4	-39.8	+3.4	+2.1	1,563.5
0.0675	3,800	54,300	400	310,000	-256,100	6,200	4,570	-7,460	-186,200	-3.70	-2.32	182.21	-39.5	-34.6	-4.9	-3.1	1,670.1
0.070	3,670	52,400	400	280,000	-228,000	6,110	4,570	-7,500	-157,800	-3.14	-1.96	164.99	-39.8	-29.4	-10.4	-6.5	1,773.6
0.0725	3,560	50,700	400	248,000	-197,700	6,040	4,570	-7,540	-127,100	-2.53	-1.58	145.81	-40.0	-23.8	-16.2	-10.1	1,870.6
0.075	3,420	48,750	400	213,000	-164,650	5,950	4,570	-7,590	-93,650	-1.86	-1.16	125.05	-40.2	-17.4	-22.8	-14.3	1,957.5
0.0775	3,300	47,200	400	175,000	-128,200	5,880	4,570	-7,640	-56,700	-1.12	-0.70	103.13	-40.5	-10.5	-30.0	-18.8	2,030.1
0.080	3,140	44,800	400	137,000	-92,600	5,830	4,570	-7,750	-20,000	-0.40	-0.25	80.51	-41.0	-3.7	-37.3	-23.3	2,083.9
0.0825	3,030	43,300	400	98,200	-55,300	5,750	4,570	-7,740	+17,200	+0.34	+0.21	57.64	-41.0	+2.0	-43.0	-26.9	2,114.4
0.085	2,960	42,500	400	59,400	-17,300	5,680	4,570	-7,750	+55,200	+1.10	+0.68	34.98	-41.0	+10.3	-51.3	-32.1	2,118.0
0.0875	2,820	40,200		60,600	-20,400	5,600	4,570			-0.30	-0.19	35.66					2,090.0
0.090	2,630	37,500		61,400	-23,900	5,510	4,570			-0.36	-0.22	36.15					2,090.0
0.0925	2,300	32,800		61,900	-29,100	5,480	4,570			-0.44	-0.27	36.42					2,090.0
0.095	2,290	32,600		61,900	-29,300	5,400	4,570			-0.44	-0.27	36.42					2,090.0
0.0975												36.15					

$$M_o = F_f y_f - F_b y_b - F_p y_p - B\theta + mg\bar{y} \sin \theta$$

$$F_o = F_f - F_b - F_p - \mu mg - \mu V + \mu m_r \omega_o^2 \bar{y}_r + m_r \omega_o^2 \bar{y}_r \sin \theta$$

The terms $\sin \theta$ and ω_o are negligibly small, hence they are neglected.

While the structure is not sliding

$$\alpha_o = \frac{M_o}{I_o} \quad \text{(eq 9.6)}$$

The numerical analysis is performed in table 9.35 from which the maximum rigid body translation $\left[x_o = 2.12(10)^{-2} \text{ ft at } t = 0.085 \text{ sec}\right]$ and the maximum rigid body rotation $\left[\theta = 2.28(10)^{-3} \text{ radians at } t = 0.0525 \text{ sec}\right]$ are determined. The rigid body translation of $x_o = 0.02$ ft and the rotation which corresponds to a vertical motion at the front of the structure equal to 0.00228(48/2) = 0.055 ft are not excessive, hence no change is anticipated in the design at this point. The values of α_o and $\ddot{x}_o$ contained in the tabulation are used to determine the rigid body inertial forces in the shear wall analysis of the following paragraph. The symbols used for column headings in table 9.35 are as follows:

$F_f - F_b$ = net lateral blast load applied to structure (table 9.34)

$F_f y_f - F_b y_b$ = moment of net lateral blast load about axis of rotation "0" = 14.25 $(F_f - F_b)$

$F_p y_p$ = moment of passive pressure soil resistance about axis of rotation = 0.83 (F_p). F_p from paragraph 9.24c

$B\theta$ = moment of earth resistance due to overturning $= \left[17.0(10)^7 \text{ ft-kips/radian}\right] (\theta)$. See paragraph 9.24b

M_o = moment of all external forces about axis of rotation $= \left[F_f y_f - F_b y_b\right] - F_p y_p - B\theta$

μV = friction resistance caused by total vertical blast load (table 9.34)

μmg = frictional resistance caused by dead load. mg from table 9.31

F_o = summation of external horizontal forces applied to the structure

$= (F_f - F_b) - F_p - \mu mg - \mu V$

$M_o - F_o y$ = numerator of equation (9.4). $y = 9.36$ ft (table 9.31)

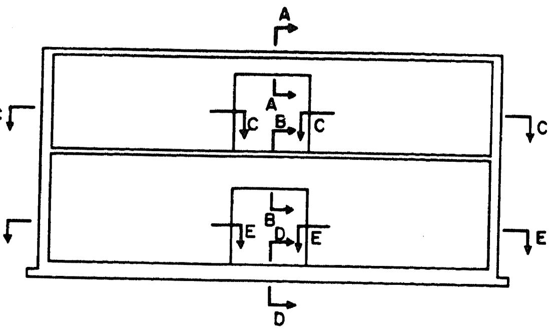
α_o = angular acceleration of structure about longitudinal axis of rotation "O" $= \dfrac{M_o - F_o\bar{y}}{I_o - m\bar{y}^2}$; where $I_o - m\bar{y}^2 = 66{,}900 - 188.96(9.37)^2$

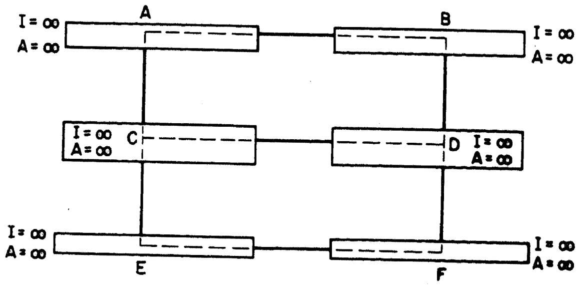
$= 50{,}300$ kip-sec^2/ft

$\ddot{x}_o$ = horizontal acceleration of axis of rotation "O" $= \dfrac{F_o}{m} - \alpha_o\bar{y}$ (eq 9.5)

9-25 DETERMINATION OF SHEAR WALL RESISTANCE FUNCTIONS. a. General. In order to perform a dynamic analysis to determine the shear wall behavior, it is necessary to determine the resistance functions for the structure. The resistance functions are expressions relating the lateral deformation of the structure to the internal resistance which the structure develops in resisting the deformation. These relations change as the deformations increase and various portions of the structure become plastic; however, the initial functions are computed for a completely elastic structure.

Since the transverse girders are relatively flexible compared to the shear walls, no rigid frame beam and column action develops, and the only resistance to lateral deformation of the structure is developed by the shear walls. Since the two end shear walls are different from the interior walls, an investigation must be made of each type in order to determine the total resistance of the entire structure. A moment and shear distribution of the shear walls will be performed in accordance with the procedures described in paragraphs 9-04 and 9-05 in order to obtain the resistance function of each wall. The resistances will then be combined in order to determine the resistance function of the structure acting as a unit. A 50 percent reduction in stiffness of the end walls is assumed because of the simultaneous blast load normal to the end wall surface. The preliminary shear wall investigation is based upon an assumed wall thickness of 10 in. which is considered the practical minimum thickness for reinforced concrete walls.





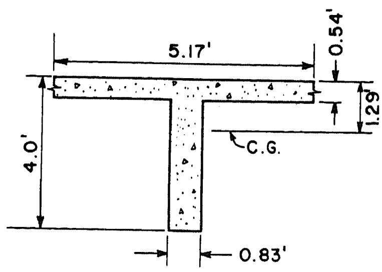
b. Interior Shear Walls. The interior shear walls have corridor openings in both stories, hence the shear wall is considered as a two-story bent. The properties of the individual members of the frame are determined so that a moment distribution may be performed to determine the resistance functions. Unit lateral distortions are introduced consecutively at the roof and upper floor levels to determine the stiffness constants which define the resistance functions. The stiffness constants are computed for the interior shear wall in paragraph 9-25o.

c. Section A-A (Lintel Section).

$$b = 8t + b' = 8(0.54) + 0.83 = 5.17 \text{ ft}$$

$$\bar{y} = \frac{4.34(0.54)^2 + 0.83(4.0)^2}{2[0.54(4.34) + 0.83(4.0)]}$$

$$= \frac{14.65}{2(5.66)} = 1.29 \text{ ft}$$

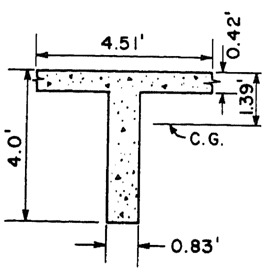


$$I_{CG} = (0.83)4.0\left[\frac{(4.0)^2}{12} + (0.71)^2\right] + (0.54)\,4.34\left[\frac{(0.54)^2}{12} + (1.02)^2\right]$$

$$= 6.06 + 2.96 = 9.02 \text{ ft}^4$$

$$A_w = (0.83)4.0 = 3.32 \text{ ft}^4$$

d. Section B-B (Lintel Section).



$$b = 8(0.42) + 0.83 = 4.51 \text{ ft}$$

$$\bar{y} = \frac{3.68(0.42)^2 + 0.83(4.0)^2}{2[0.46(3.68) + 0.83(4.0)]}$$

$$= \frac{14.078}{2(5.02)} = 1.39 \text{ ft}$$

$$I_{CG} = (0.83)4.0\left[\frac{(4.0)^2}{12} + (0.61)^2\right] + (0.46)3.68\left[\frac{(0.42)^2}{12} + (1.16)^2\right]$$

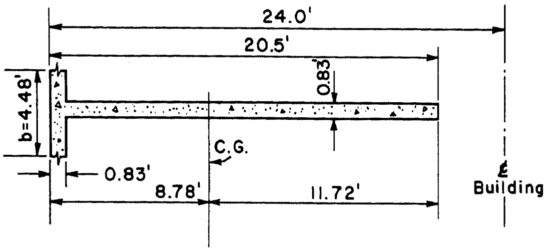
$$= 5.66 + 2.31 = 8.07 \text{ ft}^4$$

$$A_w = 0.83(4.0) = 3.32 \text{ ft}^2$$

e. Section C-C (Shear Wall Section).

$$b = 2\left[\frac{1}{6}\text{ height}\right] + t$$

$$= 2(10.96/6) + 0.83 = 4.48 \text{ ft}$$



or

$$b = 8t + b' = 8(0.83) + 0.83$$

$$= 7.46$$

$4.48 < 7.46$, hence 4.48 is controlling value for b

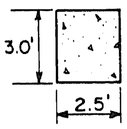
$$\bar{y} = \frac{(20.5)^2\ 0.83 + (0.83)^2\ 3.65}{2[0.83(20.5) + 0.83(3.65)]} = 8.78 \text{ ft}$$

$$I_{CG} = (20.5)0.83\left[\frac{(20.5)^2}{12} + (1.47)^2\right] + (3.65)0.83\left[\frac{(0.83)^2}{12} + (8.36)^2\right]$$

$$= 635 + 213 = 848 \text{ ft}^4$$

$$A_w = (20.5)0.83 = 17.0 \text{ ft}^2$$

f. Section D-D (Footing Section).



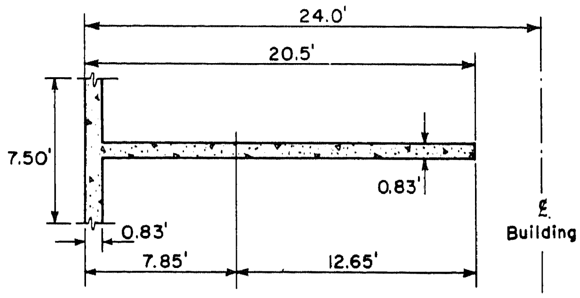
$$A = (2.50)3.0 = 7.5 \text{ ft}^2$$

$$I = \frac{7.5(3.0)^2}{12} = 5.62 \text{ ft}^4$$

g. Section E-E (Shear Wall Section).

$$b = 2\left[\frac{1}{6}\text{ height}\right] + t$$

$$= 2(22.46/6) + 0.83 = 8.32 \text{ ft}$$



or

$$b = 8t + b' = 8(0.83) + 0.83$$

$$= 7.50$$

$7.50 < 8.32$, hence 7.50 is controlling value for b'

$$\bar{y} = \frac{(20.5)^2\,0.83 + (0.83)^2\,5.67}{2[(0.83)20.5 + (0.83)6.67]} = 7.85 \text{ ft}$$

$$I_{CG} = (20.5)0.83\left[\frac{(20.5)^2}{12} + (2.40)^2\right] + (6.67)0.83\left[\frac{(0.83)^2}{12} + (6.67)^2\right]$$

$$= 696 + 246 = 944 \text{ ft}^4$$

$$A_w = (20.5)0.83 = 17.0 \text{ ft}^2$$

h. <u>Constants for Moment Distribution - Member AB (See Par. 9-04).</u>

$$L = 7.0 + 2(11.72) = 30.44 \text{ ft}$$

$$x_1 = \frac{11.72}{30.44} = 0.386$$

$$x_2 = \frac{18.72}{20.44} = 0.614$$

$$a_1 = x_2 - x_1 = (0.614 - 0.386) = 0.228$$

$$a_2 = (1/2)(x_2^{\,2} - x_1^{\,2}) = (1/2)\left[(0.614)^2 - (0.386)^2\right] = 0.114$$

$$a_3 = (1/3)(x_2^{\,3} - x_1^{\,3}) = (1/3)\left[(0.614)^3 - (0.386)^3\right] = 0.0581$$

$$S = \frac{IE}{L^2AG} = \frac{(9.02)(2.2)}{(30.44)^2(3.82)} = 0.00643$$

$$a_3' = a_1S + a_3 = 0.228(0.00643) + (0.0581) = 0.05955$$

$$C_2 = \frac{a_2 - a_3'}{a_1a_3' - (a_2)^2} = \frac{0.114 - 0.05955}{0.228(0.05955) - (0.114)^2} = 90.7$$

$$C_1 = \frac{a_3'}{a_2 - a_3'}\,C_2 = \frac{0.05955}{0.114 - 0.05955}(90.7) = 99.3$$

For symmetrical members: $K_{AB} = K_{BA} = \frac{C_1}{4}\frac{I_o}{L}$

$$C.O._{AB} = C.O._{BA} = \frac{C_2}{C_1}$$

$$\therefore K_{AB} = \frac{99.3\,(9.02)}{4\,(30.44)} = 7.35$$

$$C.O._{AB} = \frac{C_2}{C_1} = \frac{90.7}{99.3} = 0.913$$

i. <u>Constants for Moment Distribution - Member CD.</u> The centroid locations of sections C-C and E-E are slightly different because of differences in effective width of flange in bending. In order to perform a moment distribution the centroid locations for C-C and E-E are averaged to obtain a common location for these sections and a consistent set of stiffness and carry-over factors for section B-B.

$$L = \frac{7.0 + 2(11.72) + 7.0 + 2(12.65)}{2} = 31.37 \text{ ft}$$

$$x_1 = \frac{[31.37 - 7.0]/2}{31.37} = \frac{12.18}{31.37} = 0.389$$

$$x_2 = \frac{12.18 + 7.0}{31.37} = 0.611$$

$$a_1 = x_2 - x_1 = 0.611 - 0.389 = 0.222$$

$$a_2 = (1/2)\left(x_2^2 - x_1^2\right) = (1/2)\left[(0.611)^2 - (0.389)^2\right] = 0.110$$

$$a_3 = (1/3)\left(x_2^3 - x_1^3\right) = (1/3)\left[(0.611)^3 - (0.389)^3\right] = 0.0564$$

$$S = \frac{IE}{L^2AG} = \frac{(8.07)\ 2.2}{(31.37)^2\ 3.32} = 0.00544$$

$$a'_3 = a_1S + a_3 = 0.222(0.00544) + 0.0564 = 0.0576$$

$$C_2 = \frac{a_2 - a'_3}{a_1a'_3 - (a_2)^2} = \frac{0.110 - 0.0576}{0.222(0.0576) - (0.110)^2} = 77.1$$

$$C_1 = \frac{a'_3}{a_2 - a'_3}\ C_2 = \frac{0.0576}{0.110 - 0.0576}\ 77.1 = 84.6$$

For symmetrical member CD:

$$K_{CD} = K_{CD} = \frac{C_1}{4}\frac{I_o}{L}$$

$$C.O._{CD} = C.O._{DC} = \frac{C_2}{C_1}$$

$$K_{CD} = \frac{84.6(8.07)}{4(31.37)} = 5.44$$

$$C.O._{CD} = \frac{C_2}{C_1} = \frac{77.0}{84.6} = 0.910$$

For antisymmetrical loading, K factors can be modified so that distribution can be performed on one-half of structure.

$K' = K(1 + C.O.)$

$K'_{AB} = (7.35)(1 + 0.912) = 14.05$

$K'_{CD} = (5.44)(1 + 0.910) = 10.30$

j. <u>Constants for Moment Distribution - Member AC.</u>

$L = 7.5 + 1.39 + 2.71 = 11.60$ ft

$$x_1 = \frac{1.39}{11.60} = 0.120$$

$$x_2 = \frac{8.89}{11.60} = 0.767$$

$$a_1 = x_2 - x_1 = 0.767 - 0.120 = 0.647$$

$$a_2 = (1/2)\left(x_2{}^2 - x_1{}^2\right) = (1/2)\left[(0.767)^2 - (0.120)^2\right] = 0.2863$$

$$a_3 = (1/3)\left(x_2{}^3 - x_1{}^3\right) = (1/3)\left[(0.767)^3 - (0.120)^3\right] = 0.150$$

$$S = \frac{IE}{L^2AG} = \frac{(848)(2.2)}{(11.60)^2\ 17.0} = 0.815$$

$$a'_3 = a_1S + a_3 = 0.647(0.815) + 0.150 = 0.677$$

$$C_2 = \frac{a_2 - a'_3}{a_1a'_3 - (a_2)^2} = \frac{0.2863 - 0.677}{0.647(0.677) - (0.2863)^2} = -1.105$$

$$C_1 = \frac{a'_3}{a_2 - a'_3}\ C_2 = \frac{0.677}{0.2863 - 0.677}\ (-1.105) = 1.92$$

$$C_3 = \frac{a_1 - 2a_2 + a'_3}{a_2 - a'_3}\ C_2 = \frac{0.647 - 2(0.2863) + 0.677}{0.2863 - 0.677}\ (-1.105) = 2.12$$

$$K_{CA} = \frac{C_1I}{4L} = \frac{1.92(848)}{4(11.60)} = 35.1$$

$$K_{AC} = \frac{C_3I}{4L} = \frac{2.12(848)}{4(11.60)} = 38.8$$

$$C.O._{CA} = \frac{C_2}{C_1} = \frac{-1.105}{1.92} = -0.576$$

$$C.O._{AC} = \frac{C_2}{C_3} = \frac{-1.105}{2.14} = -0.516$$

k. <u>Constants for Moment Distribut</u>

$$L = 7.0 + 2(12.65) = 32.30$$

$$x_1 = \frac{12.65}{32.30} = 0.392$$

$$x_2 = \frac{19.65}{32.30} = 0.608$$

$$a_1 = x_2 - x_1 = 0.608 - 0.392 = 0.$$

$$a_2 = (1/2)\left(x_2^{\,2} - x_1^{\,2}\right) = (1/2)\ \left[(0\right.$$

$$a_3 = (1/3)\left(x_2^{\,3} - x_1^{\,3}\right) = (1/3)\ \left[(0\right.$$

$$S = \frac{IE}{L^2 AG} = \frac{5.62\ (2.2)}{(32.3)^2\ 7.5} = 0.00158$$

$$a_3' = a_1 S + a_3 = 0.216(0.00158) + 0.0545 = 0.0548$$

$$C_2 = \frac{a_2 - a_3'}{a_1 a_3' - (a_2)^2} = 107.0$$

$$C_1 = \frac{a_3'}{a_2 - a_3'}\, C_2 = \frac{0.0548}{0.1085 - 0.0548}\ 107.0 = 109.5$$

$$K_{EF} = \frac{C_1 I}{4L} = \frac{109.5(5.62)}{4(32.30)} = 4.76$$

$$C.O._{EF} = \frac{C_2}{C_1} = \frac{107.0}{109.5} = 0.978$$

Modification for antisymmetrical loading:

$$K' = K(1 + C.O.) = 4.76(1 + 0.978) = 9.42$$

l. <u>Constants for Moment Distribution - Member EC.</u>

$$L = 1.5 + 7.5 + 2.61 = 11.61 \text{ ft}$$

$$x_1 = \frac{1.5}{11.61} = 0.129$$

$$x_2 = \frac{9.00}{11.61} = 0.775$$

$$a_1 = x_2 - x_1 = 0.775 - 0.129 = 0.646$$

$$a_2 = (1/2)\left(x_2{}^2 - x_1{}^2\right) = (1/2)\left[(0.775)^2 - (0.129)^2\right] = 0.2922$$

$$a_3 = (1/3)\left(x_2{}^3 - x_1{}^3\right) = (1/3)\left[(0.775)^3 - (0.129)^3\right] = 0.1546$$

$$S = \frac{IE}{L^2 AG} = \frac{(944)2.2}{(11.61)^2\ 17.0} = 0.905$$

$$a_3' = a_1 S + a_3 = 0.646(0.905) + 0.1546 = 0.7396$$

$$C_2 = \frac{a_2 - a_3'}{a_1 a_3' - (a_2)^2} = \frac{0.2922 - 0.7396}{0.646(0.7396) - (0.2922)^2} = -1.14$$

$$C_1 = \frac{a_3'}{a_2 - a_3'} C_2 = \frac{0.7396}{0.2922 - 0.7396}(-1.14) = 1.89$$

$$C_3 = \frac{a_1 - 2a_2 + a_3'}{a_2 - a_3'} C_2 = \frac{0.646 - 2(0.2922) + 0.7396}{0.2922 - 0.7396}(-1.14) = 2.04$$

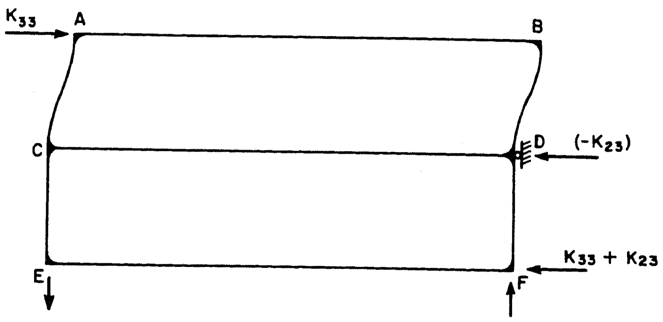
$$K_{EC} = \frac{C_1 I}{(4)(L)} = \frac{1.89(9.44)}{4(11.61)} = 38.4$$

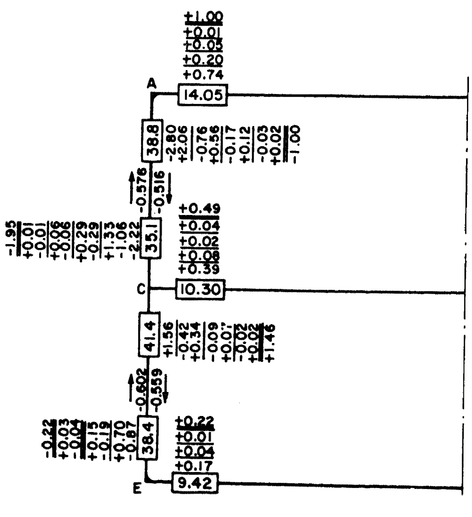
$$K_{CE} = \frac{C_3 I}{(4)(L)} = \frac{2.04(944)}{4(11.61)} = 41.4$$

$$C.O._{EC} = C_2/C_1 = -1.14/1.89 = -0.602$$

$$C.O._{CE} = C_2/C_3 = -1.14/2.04 = -0.559$$

m. Moment Distribution for Unit Lateral Displacement, Case I. Consider a unit lateral displacement of top-story member AB and compute the corresponding moments.





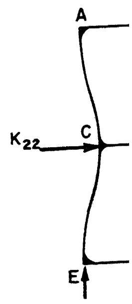
$$FEM_{CA} = -4E(1 + C.O._{CA})K_{CA}\ \psi_{CA}$$

$$= -(4)3(10)^3\ [1 + (-0.576)]\ (35$$

$$FEM_{AC} = -4E(1 + C.O._{AC})K_{AC}\ \psi_{AC}$$

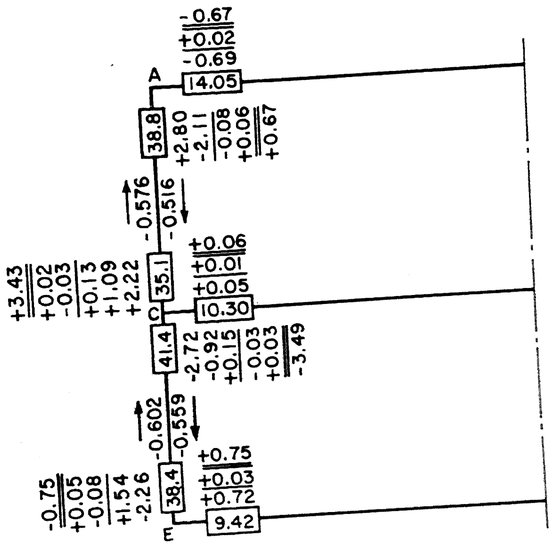
$$= -(4)3(10)^3\ [1 + (-0.516)]\ (38$$

n. Moment Distribution for Unit Lateral Displacement, Case II. Consider a unit lateral displacement of member CD and compute the corresponding moments.



$$FEM_{EC} = -4E(1 + C.O._{EC})K_{EC}\ \psi_{EC}$$

$$= -(4)3(10)^3\ [1 + (-0.602)]\ (3\varepsilon$$



FEM_{CE}

$$= -(4)3(10)^3[1 + (-0.559)](41.4)144/11.61$$

$$= -2.72(10)^6 \text{ ft-kips}$$

o. Summary of Resistances - Interior Shear Walls. From the first distribution (assume positive direction to right):

$$K_{33} = 2(1.95 + 1.00)(10)^6/11.60$$

$$= 509{,}000 \text{ kips/ft}$$

$$-K_{23} = \left[2(1.95 + 1.00(10)^6/11.60\right] + \left[2(1.46 - 0.22)(10)^6/11.61\right] = +722{,}000 \text{ kips/ft}$$

From the second distribution:

$$K_{22} = \left[2(0.75 + 3.49)(10)^6/11.61\right] + \left[2(3.43 + 0.67)(10)^6/11.60\right]$$

$$= 730{,}000 + 706{,}000 = 1{,}436{,}000 \text{ kips/ft}$$

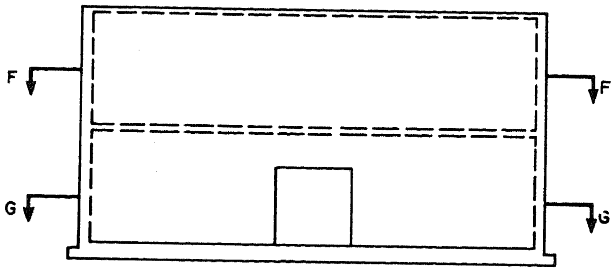
$$-K_{32} = 2(3.43 + 0.67)(10)^6/11.60 = 706{,}000 \text{ kips/ft}$$

Combining the two moment distribution solutions:

$$R_3 = K_{33}x_3 + K_{32}x_2 = 509{,}000\ x_3 - 706{,}000\ x_2$$

$$R_2 = K_{22}x_2 + K_{23}x_3 = 1{,}436{,}000\ x_2 - 722{,}000\ x_3$$

The expressions for R_3 and R_2 given here express the resistance developed by the interior shear wall as a function of the deflections x_3 and x_2 at the roof and upper floor levels, respectively.



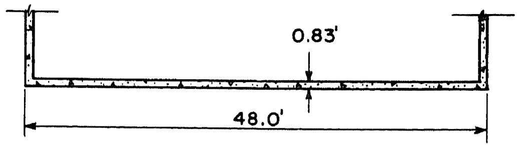
p. End Shear Walls. The end shear walls have corridor openings only in the lower story, hence only the lower story is analyzed as a bent. However, the shear deformation in the upper story must be included in the shear wall analysis. The properties of the individual members are determined so that the resistance functions may be evaluated.

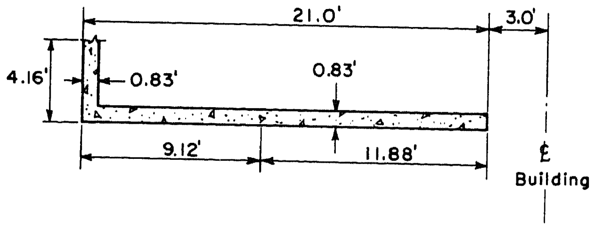
q. Section F-F.

$h = 11.50 - 0.54 = 10.96$

= clear end wall height in second story

$A = (0.83)(48.0) = 40.0\ ft^2$





r. Section G-G.

Effective flange width

$b = (h/6) + b' = (22.46/6) + 0.83 = 4.58\ ft$

or $b = 4t + b' = 4(0.83) + 0.83 = 4.16\ ft$

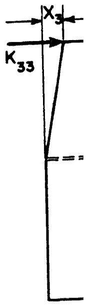
$4.16 < 4.58$ ft, hence $b = 4.16$

$$\bar{y} = \frac{(21.0)^2\ 0.83 + (0.83)^2\ 3.33}{2\left[0.83(21.0) + 0.83(3.33)\right]} = 9.12\ ft$$

$$I_{CG} = (21.0)0.83\left[\frac{(21.0)^2}{12} + (1.38)^2\right] + (3.33)0.83\left[\frac{(0.83)^2}{12} + (3.33)^2\right]$$

$$= 678 + 308 = 1{,}086\ ft^4$$

$$A_w = (21.0)0.83 = 17.5\ ft^2$$



s. Method of Analysis - Upper Story. Since the upper story of the end walls has no opening, it will be necessary to analyze the end walls in a slightly different manner from the interior walls. For the first case, the holding force will be applied at the second floor level and tl roof level.

Since in the previous analyses the been considered in the moment distribut: the analysis.

In this case:

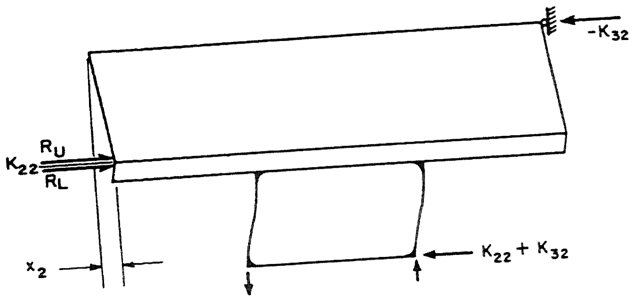
$$x = \frac{RH}{AG} \; ; \quad \frac{E}{G} = 2.2 \; ; \quad G = \frac{E}{2.2}$$

$$R = \frac{AG}{H} x_3 = \frac{(40.0)3(10)^3\ 144(1.0)}{(10.96)2.2}$$

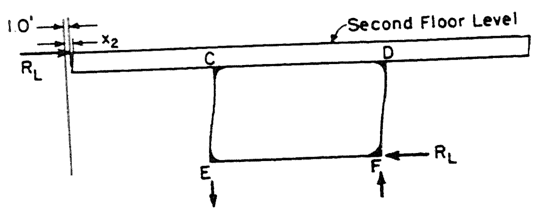
$$R = 716{,}000 \text{ kips}$$

$$K_{33} = R = 716{,}000 \text{ kips/ft}$$

$$-K_{23} = +R = 716{,}000 \text{ kips/ft}$$



t. Method of Analysis - Lower Story. For the second case the distorting force will be applied at the second floor level and the holding force at the roof level. For simplification in analysis, the distorting force can be visualized as consisting of two forces--one acting to distort the top story in a similar manner to that of the top story discussed above, and the other force acting on the lower story causing distortion of the lower story members by frame action.



For $x_2 = 1.0$ ft

$$R_u = \frac{AG}{H} x_2 = \frac{(40.0)3(10)^3\ 144(1.0)}{(10.96)2.2} = 716{,}000 \text{ kips}$$

u. Constants for Moment Distribution - Member CE.

$L = 7.5 + 1.5 = 9.0$ ft

$$x_1 = \frac{1.5}{9.0} = 0.1665$$

$$x_2 = \frac{9.0}{9.0} = 1.0$$

$$a_1 = x_2 - x_1 = 1.0 - 0.1665 = 0.8335$$

$$a_2 = (1/2)\left(x_2^{\,2} - x_1^{\,2}\right) = (1/2)\left[(1.0)^2 - (0.1665)^2\right] = 0.4861$$

$$a_3 = (1/3)\left(x_2^{\,3} - x_1^{\,3}\right) = (1/3)\left[(1.0)^2 - (0.1665)^3\right] = 0.3318$$

$$S = \frac{IE}{L^2 AG} = \frac{(1{,}086)2.2}{(9.0)^2 17.5} = 1.685$$

$$a_3' = a_1 S + a_3 = 0.8335(1.685) + 0.3318 = 1.737$$

$$C_2 = \frac{a_2 - a_3'}{a_1 a_3' - (a_2)^2} = \frac{0.4861 - 1.737}{0.8335(1.737) - (0.4861)^2} = -1.128$$

$$C_1 = \frac{a_3'}{a_2 - a_3'} C_2 = \frac{1.737}{0.4861 - 1.737}(-1.128) = 1.565$$

$$C_3 = \frac{a_1 - 2a_2 + a_3'}{a_2 - a_3'}(C_2) = \frac{0.8335 - 2(0.4861) + 1.737}{0.4861 - 1.737}(-1.128) = 1.52$$

$$K_{CE} = \frac{C_3 I}{4L} = \frac{152(1{,}086)}{4(9.0)} = 45.9$$

$$C.O._{CE} = \frac{C_2}{C_3} = \frac{-1.128}{1.52} = -0.742$$

$$K_{EC} = \frac{C_1 I}{4L} = \frac{1.565(1{,}086)}{4(8.5)} = 50.1$$

$$C.O._{EC} = \frac{C_2}{C_1} = \frac{-1.128}{1.565} = -0.722$$

v. Constants for Moment Distribution - Member EF.

$L = 2(11.88) + 6.0 = 29.76$

$$x_1 = \frac{11.88}{29.76} = 0.400$$

$$x_2 = \frac{17.88}{29.76} = 0.600$$

$$a_1 = x_2 - x_1 = (0.600 - 0.400) = 0.200$$

$$a_2 = (1/2)\left(x_2^{\,2} - x_1^{\,2}\right) = (1/2)\left[(0.600)^2 - (0.400)^2\right] = 0.1000$$

$$a_3 = (1/3)\left(x_2^{\,3} - x_1^{\,3}\right) = (1/3)\left[(0.600)^3 - (0.400)^3\right] = 0.05066$$

$$S = \frac{IE}{L^2 AG} = \frac{(5.62)2.2}{(29.76)^2\ 7.5} = 0.00186$$

$$a_3' = a_1 S + a_3 = 0.200(0.00186) + 0.05066 = 0.05103$$

$$C_2 = \frac{a_2 - a_3'}{a_1 a_3' - (a_2)^2} = \frac{(0.100 - 0.05103)}{0.200(0.05103) - (0.100)^2} = 216.0$$

$$C_1 = \frac{a_3'}{a_2 - a_3'} C_2 = \frac{0.05103}{0.100 - 0.0510}(216) = 226.0$$

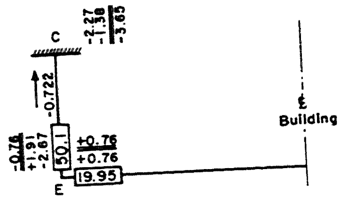
$$K_{EF} = \frac{C_1 I}{4L} = \frac{216(5.62)}{4(29.76)} = 10.2$$

$$C.O._{EF} = \frac{C_2}{C_1} = \frac{216.0}{226.0} = 0.955$$

Modification for antisymmetrical loading:

$$K_{EF}' = (1 + C.O.)K = (1 + 0.955)10.2 = 19.95$$

w. <u>Moment Distribution for Unit Lateral Displacement - Lower Story.</u>



$$FEM_{EC} = -4E(1 + C.O._{EC})K_{EC}\,\psi_{EC}$$

$$= -(4)3(10)^3[1 + (-0.722)](50.1)144(1.0)/9.0$$

$$= -2.67(10)^6 \text{ ft-kips}$$

$$FEM_{CE} = -4E(1 + C.O._{CE})K_{CE}\,\psi_{CE}$$

$$= -(4)3(10)^3[1 + (-0.742)](45.9)144(1.0)/9.0$$

$$= -2.27(10)^6 \text{ ft-kips}$$

$$R_L' = 2(3.65 + 0.76)(10)^6/9.0 = 980{,}000 \text{ kips/ft}$$

Taking into account shear distortion above opening on lower story

$$x_2' = \frac{RH}{AG} = \frac{(980{,}000)4.0(2.2)}{0.83(48.0)3(10)^3 144} = 0.497 \text{ ft}$$

$$x_2 = 1.0 \text{ ft} + x_2' = 1.0 + 0.497 = 1.497 \text{ ft}$$

Thus for a one-foot deflection at the second floor level

$$R_L = \frac{980{,}000}{1.497} = 655{,}000 \text{ kips/ft}$$

Combining lower and upper distortion forces

$$K_{22} = F_{2U} + F_{2L} = 716{,}000 + 655{,}000 \text{ kips/ft} = 1{,}371{,}000 \text{ kips/ft}$$

$$-K_{32} = -F_{2U} = -716{,}000 \text{ kips/ft}$$

x. Summary of Resistances - End Shear Wall.

Combining distributions:

$$R_3 = K_{33}x_3 + K_{32}x_2 = 716{,}000\ x_3 - 716{,}000\ x_2$$

$$R_2 = K_{22}x_2 + K_{23}x_3 = 1{,}371{,}000\ x_2 - 716{,}000\ x_3$$

y. Summary of Total Resistances of Entire Building. The entire building consists of four interior shear walls plus the two end shear walls. In order to perform a shear wall analysis on the entire building, the values are combined below taking into account a reduction of one-half for the resistance of the end shear walls.

$$R_3 = \left[4(509{,}000) + \tfrac{2}{2}(716{,}000)\right] x_3 - \left[4(706{,}000) + \tfrac{2}{2}(716{,}000)\right] x_2$$

$$R_3 = 2{,}752{,}000\ x_3 - 3{,}540{,}000\ x_2$$

$$R_2 = \left[4(1{,}436{,}000) + \tfrac{2}{2}(1{,}371{,}000)\right] x_2 - \left[4(722{,}000) + \tfrac{2}{2}(716{,}000)\right] x_3$$

$$= 7{,}115{,}000\ x_2 - 3{,}604{,}000\ x_3$$

9-26 SHEAR WALL ANALYSIS. Utilizing the resistance functions obtained in paragraph 9-25 (table 9.39, page 153), a dynamic analysis was performed in order to determine the lateral displacements of the shear walls. These displacements are necessary to obtain shear and moment values in the wall elements so that proper reinforcement can be provided (par. 9-29). The

shear wall analysis also determines specific resistance values which are utilized in the deep beam analysis of roof and floor slab design (par. 9-28) and wall design (par. 9-29).

a. Computation of Masses m_2 and m_3. The division of the total structure mass into three separate parts for the shear wall analysis is made in the following manner: the division between m_1 and m_2 occurs halfway between the top of the footing and the underside of the second floor; the division between m_2 and m_3 occurs halfway between the top of the second floor and the underside of roof slab. The masses m_3 and m_2 are computed in tables 9.36 and 9.37, respectively, and m_1 is obtained by subtraction from the total mass.

Table 9.36. Computation of m_3

Element	Dimensions (or Ratio of Total)	Total Mass (kip-sec^2/ft)	Volume (cu ft)	Weight (kips)	Mass (kip-sec^2/ft)	$\bar{y}$* (ft)	$m\bar{y}$
Front wall	6.02/24.64	15.05			3.67	3.01	11.1
Rear wall	6.02/24.64	15.05			3.67	3.01	11.1
End walls	2(0.83)(46.3)(5.48)						
Shear walls	4 [(0.83)(46.3)(5.48) - (1.48)(7.0)]						
Roof slab	(1.0)	18.90					
Transverse roof girders	(1.0)	1.89					
Longitudinal roof girders	(1.0)	1.44					
2nd floor columns	4.52/10.0	1.68					
					36.06		52.6

$\bar{y}_{\text{from top}} = \frac{52.6}{36.06} = 1.46$ ft

* From top of m_3.

Table 9.37. Computation of m_2

Element	Dimensions (or Ratio of Total)	Total Mass (kip-sec^2/ft)	Volume (cu ft)	Weight (kips)	Mass (kip-sec^2/ft)	$\bar{y}$* (ft)	$m\bar{y}$
Front wall	12.54/24.64	15.05			7.65	6.27	47.9
Rear wall	12.54/24.64	15.05			7.65	6.27	47.9
End walls	2 [(0.83)46.3(12.54) - (6.0)(3.10)]		931	140	4.35	6.27	27.3
Shear walls	4 [(0.83)46.3(12.08) - (7.0)(8.58)]		1,628	244	7.57	6.27	47.4
Floor slab	(1.0)	15.40			15.40	5.71	88.0
2nd floor trans girders	(1.0)	1.42			1.42	6.44	9.2
2nd floor long. girders	(1.0)	1.46			1.46	6.44	9.4
2nd floor columns	(5.48)/10.0	1.68			0.92	2.74	2.5
2nd floor occupancy	(1.0)	3.35			3.35	4.48	15.0
1st floor columns	5.60/(12.0)	2.01			0.94	9.74	9.2
					50.71		303.8

$\bar{y}* = \frac{303.8}{50.71} = 5.97$ ft

* From top of m_2.

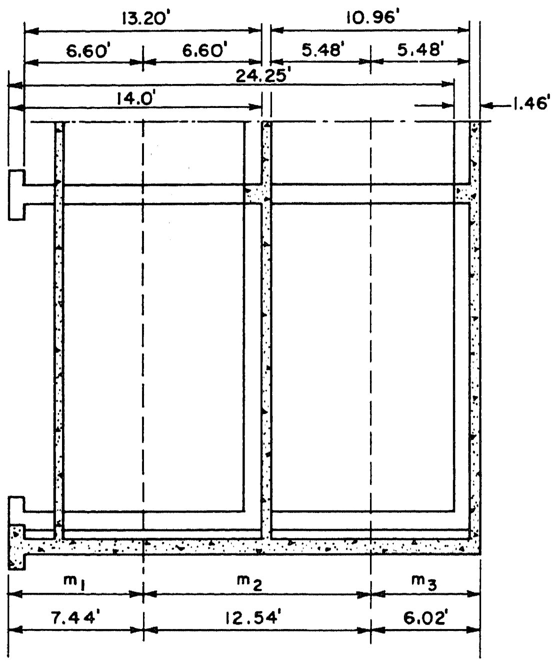
$m_2 = 50.71$ kip-sec^2/ft

$m_3 = 36.06$

$m_1 = m - m_2 - m_3 = 188.96 - 50.71 - 36.06 = 102.19$ kip-sec^2/ft

The location of the centroid of each mass (m_1, m_2, and m_3) is also computed (tables 9.36 and 9.37) for further use in the determination of accelerations of masses m_2 and m_3, and for use in a dynamic simultaneous overturning and sliding analysis.

Measuring distances from the base of the footing



$\bar{y}_3 = 26.0 - 1.46 = 24.54$ ft

$\bar{y}_2 = 7.43 + 12.54 - 5.97$

$= 14.0$ ft

$m_1\bar{y}_1 + m_2\bar{y}_2 + m_3\bar{y}_3 = m\bar{y}$

$$\bar{y}_1 = \frac{m\bar{y} - m_2\bar{y}_2 - m_3\bar{y}_3}{m_1}$$

$m\bar{y} = 1{,}770$ (table 9.31)

$m_2\bar{y}_2 = (50.71)14.0 = 712$

$m_3\bar{y}_3 = (36.06)24.54 = 885$

$$y_1 = \frac{1{,}770 - 712 - 885}{102.19}$$

$= 1.69$ ft

b. Computation of Accelerations $\ddot{x}_2$ and $\ddot{x}_3$. Using the accelerations $\ddot{x}_o$ and α_o obtained from the rigid body overturning and sliding analysis (table 9.35) the acceleration at m_2 and m_3 will be obtained for use in a preliminary shear wall analysis.

$\ddot{x}_2 = \ddot{x}_o + 14.0\alpha_o$

$\ddot{x}_3 = \ddot{x}_o + 24.54\alpha_Q$

c. Dynamic Analysis. The shear wall behavior is obtained by performing two simultaneous dynamic analyses by numerical integration. These dynamic analyses are performed in table 9.39. The column headings and notations used in this table are outlined below. Various quantities are computed in the dynamic analysis to determine the points at which the resistance functions should be modified. This is accomplished in

Table 9.38. *Acceleration of m_2 and m_3*

t (sec)	$\ddot{x}_o$ (ft/sec^2)	α_o (radians/sec^2)	$\ddot{x}_2$ (ft/sec^2)	$\ddot{x}_3$ (ft/sec^2)
0	19.9	1.25	37.4	50.6
0.0025	22.8	2.52	58.1	84.8
0.005	18.2	2.47	52.8	78.8
0.0075	13.8	2.38	47.1	72.3
0.010	12.5	2.19	43.2	66.3
0.0125	6.8	2.06	35.6	57.4
0.015	3.6	1.84	29.4	48.8
0.0175	1.2	1.58	23.3	39.5
0.0200	-1.3	1.28	16.6	30.1
0.0225	-3.2	0.97	10.4	20.6
0.025	-5.13	0.62	3.57	10.07
0.0275	-6.67	0.27	-2.89	-0.03
0.030	-2.55	-0.73	-12.75	-20.45
0.0325	-6.90	-1.12	-22.60	-34.4
0.035	-7.9	-1.78	-32.80	-51.5
0.0375	+3.1	-2.55	-32.6	-59.4
0.040	+6.8	-3.24	-38.6	-72.7
0.0425	+12.6	-3.88	-41.6	-82.6
0.045	+11.5	-4.24	-47.9	-92.5
0.0475	+12.7	-4.62	-51.8	-100.3
0.050	+13.3	-4.92	-55.5	-107.2
0.0525	+11.8	-5.06	-58.8	-112.2
0.055	+10.0	-5.11	-61.5	-115.5
0.0575	+8.7	-5.04	-61.8	-115.3
0.060	+6.8	-4.86	-61.2	-112.2
0.0625	+4.0	-4.55	-59.6	-107.5
0.065	+3.4	-4.26	-56.2	-101.1
0.0675	-4.9	-3.70	-56.7	-95.5
0.070	-10.4	-3.14	-54.3	-87.4
0.0725	-16.2	-2.53	-51.6	-78.2
0.075	-22.8	-1.86	-48.8	-68.5
0.0775	-30.0	-1.12	-45.7	-57.5
0.080	-37.3	-0.40	-43.1	-47.1
0.0825	-43.0	+0.34	-38.3	-34.7
0.085	-51.3	+1.10	-35.9	-25.30
0.0875	0	-0.30	-4.20	-7.36
0.090	0	-0.36	-5.04	-8.84
0.0925	0	-0.44	-6.16	-10.80
0.095	0	-0.44	-6.16	-10.80

table 9.40 which serves as a work sheet for table 9.39. The resistance functions necessary for table 9.39 are computed in paragraphs 9-25 and 9-26e, f, g, h, i, j.

Using acceleration impulse extrapolation method (par. 5-08d) the following equations define the numerical integration operations:

$$(x_2)_{t = n + 1} = 2(x_2)_{t = n} - (x_2)_{t = n - 1} + (\ddot{x}_{2\ net})_{t = n} (\Delta t)^2$$
$$(x_3)_{t = n + 1} = 2(x_3)_{t = n} - (x_3)_{t = n - 1} + (\ddot{x}_{3\ net})_{t = n} (\Delta t)^2$$

Use $\Delta t = 0.0025$ sec (same time interval used for rigid body analysis)

where

$\ddot{x}_{2\ net}$ = relative lateral acceleration of mass $m_2 = \dfrac{P_2 - R_2}{m_2} - \ddot{x}_2$

$\ddot{x}_{3\ net}$ = relative lateral acceleration of mass $m_3 = \dfrac{P_3 - R_3}{m_3} - \ddot{x}_3$

P_2 = blast load on mass m_2 consisting of net lateral wall slab reactions obtained from dynamic analyses of front and back wall slabs (tables 9.29 and 9.30) = $156.8(2)(V_{2n} - V_2)$

P_3 = blast loads on mass m_3 consisting of net lateral wall slab reactions obtained from dynamic analyses of front and back wall slabs (tables 9.29 and 9.30) = $156.8\ (V_{1n} - V_1)$

R_2 = resistance function, expressing resistance of the structure acting upon m_2 in terms of displacement of masses m_2 and m_3. This resistance function will change as various shear wall members exceed their elastic stresses. For completely elastic behavior, $R_2 = 71.15(10)^5\ x_2 - 36.01(10)^5\ x_3$ (par. 9-25y). At time t = 0.025, members AB, CD, and wall element CE of the interior shear wall become plastic, and $R_2 = 5{,}900 + 13.71(10)^5\ x_2 - 7.16(10)^5\ x_3$. At time t = 0.030, the first-story members of the end shear wall become plastic, and $R_2 = 10{,}020 + 7.16(10)^5\ x_2 - 7.16(10)^5\ x_3$. At time t = 0.050, the first-story shear wall members in both interior and end shear walls return to the elastic range, and $R_2 = -15{,}310 + 68.31x_2 - 39.46x_3$. At time t = 0.0550, the first-story wall members are again in the plastic range, hence $R_2 = 4{,}350 + 7.16(10)^5\ x_2 - 7.16(10)^5\ x_3$.

R_3 = resistance function, expressing resistance of the structure acting upon m_3, in terms of displacements of m_2 and m_3. This function will change as various shear wall members exceed their elastic stresses. For completely elastic deflection, $R_3 = 27.52(10)^5\, x_3 - 35.4(10)^5\, x_2$. At time t = 0.025, members AB, CD, and wall element CE of the interior shear wall become plastic, and $R_3 = 2{,}180 + 7.16(10)^5\, x_3 - 7.16(10)^5\, x_2$. At time t = 0.030, the first-story members of the end shear wall become plastic, and $R_3 = 2{,}160 + 7.16(10)^5\, x_3 - 7.16(10)^5\, x_2$. At time t = 0.050, the first-story shear wall members in both interior and end shear walls return to the elastic range, and $R_3 = 8{,}346 + 24.46\, x_3 - 32.86\, x_2$. At time t = 0.0550, the first-story wall members are again in the plastic range, hence $R_3 = 4{,}876 + 7.16\, x_3 - 7.16\, x_2$.

The dynamic analysis is presented in table 9.39. The terms used as column headings in the tabulation are discussed below:

V_c = net lateral slab reaction at top of second floor wall slab obtained by subtracting V_1 value (table 9.30) from V_{1n} value (table 9.29).

V_d = blast pressure on edge of roof slab over wall

$$\frac{0.54(144)}{1{,}000} (\overline{P}_{front} - \overline{P}_{back}) = 0.077\, (\overline{P}_{front} - \overline{P}_{back})$$

$(\overline{P}_{front} - \overline{P}_{back}$ from figure 9.34)

P_3 = total blast load on mass m_3 at top of shear wall

$= 156.8\, (V_c + V_d)$

V_e = net lateral reaction at second floor level, consisting of $2(V_{2n} - V_2)$. These values are obtained from tables 9.29 and 9.30, respectively.

V_f = blast pressure on edge of second floor slab

$$\frac{0.46(144)}{1{,}000} (\overline{P}_{front} - \overline{P}_{back}) = 0.066\, (\overline{P}_{front} - \overline{P}_{back})$$

P_2 = total blast load on mass m_2 at top of shear wall

$= 156.8\, (V_e - V_f)$

R_3 = resistance function (par. 9-25y)

$\frac{P_3 - R_3}{m_3}$ = acceleration of mass m_3 without considering rotation of building

$\ddot{x}_3$ = acceleration of top of shear wall from rigid body overturning analysis (table 9.38)

$\ddot{x}_{3\ net}$ = relative acceleration of mass m_3 $= \frac{P_3 - R_3}{m_3} - \ddot{x}_3$

x_3 = displacement of mass m_3

$\frac{P_2 - R_2}{m_2}$ = acceleration of mass m_2 without considering rotation of building

$\ddot{x}_2$ = acceleration of mass m_2 from rigid body overturning analysis (table 9.38)

$\ddot{x}_{2\ net}$ = relative acceleration of mass m_2

x_2 = displacement of mass m_2

$(R_2 + R_3)$ = load on first-story shear wall panels

Maximum value for R_3(7,046 kips) occurs at t = 0.0550 sec

Maximum value for R_2(9,853 kips) occurs at t = 0.0450 sec

From the analysis in table 9.39 the maximum second story relative deflection ($x_3 - x_2$ = 0.00435 ft) is reached at time t = 0.0325 sec and the maximum first story relative deflection (x_2 = 0.02195 ft) is reached at time t = 0.085 sec. These correspond with deflections of $x_3 - x_2$ = 0.00310 ft and x_2 = 0.00520 at time t = 0.0250 sec when the first cracking of the shear walls occurs (par. 9-26d). The allowable maximum deflection as a function of the deflection at the time when a wall first cracks is given by equation (4.53) $\left[\delta_u = 24 \frac{H}{L} \delta_c = 24 \left(\frac{11.6}{18.5}\right) \delta_c = 15\ \delta_c\right]$. As this criteria is satisfied $\left(\frac{0.00435}{0.00310} = 1.4 < 15 \text{ and } \frac{0.02195}{0.00520} = 4.2 < 15\right)$ the 10-in. shear walls which are assumed for the preliminary analysis are satisfactory.

d. Modification of Resistance Functions for Table 9.39. In the analysis of the shear walls, the resistance of the various members will not increase after the maximum shear stress is exceeded. Table 9.40, which is

Table 9.39. Preliminary Shear Wall Analysis

t (sec)	V_3: V_c	V_3: V_s	P_3 (kips)	V_2: V_e	V_2: V_r	P_2 (kips)	R_3 (kips)	$\frac{P_3 - R_3}{M_3}$	$\ddot{x}_3$ (ft/sec²)	$\ddot{x}_3$ net (ft/sec²)	$\ddot{x}_3$ net $(\Delta t)^2$ (10^{-5} ft)	x_3 (10^{-5} ft)	$\frac{P_2 - R_2}{M_2}$	$\ddot{x}_2$ (ft/sec²)	$\ddot{x}_2$ net (ft/sec²)	$\ddot{x}_2$ net $(\Delta t)^2$ (10^{-5} ft)	x_2 (10^{-5} ft)	R_2 (kips)	$P_2 + P_3$ (kips)
0	5.4	2.2	[illegible]	[illegible]	[illegible]	1,000	0	[illegible]	[illegible]	[illegible]	[illegible]	0	[illegible]	[illegible]	[illegible]	[illegible]	0	0	0
0.0025	7.6	2.2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.005	15.9	2.1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.0075	20.4	2.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.010	20.3	2.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.0125	20.1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.015	20.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.0175	19.8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.020	19.6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.0225	19.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.025	19.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.0275	19.2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.030	19.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.0325	18.8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.035	12.1	1.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.0375	9.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.040	8.8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.0425	8.2	1.1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.045	7.5	1.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.0475	6.9	1.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.050	6.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.0525	5.6	0.8	1,000	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.055	5.0	0.8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.0575	5.0	0.7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.060	4.9	0.7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.0625	4.7	0.6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.065	4.5	0.6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.0675	4.4	0.6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	9,228
0.070	4.2	0.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1,887.2	[illegible]	9,228
0.0725	4.0	0.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	6,220	9,228
0.075	3.8	0.5	674	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2,074.2	6,040	9,228
0.0775	3.7	0.5	660	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2,135.2	5,760	9,228
0.080	3.5	0.5	628	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2,175.9	5,470	9,228
0.0825	3.4	0.4	596	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2,195.1*	5,270	9,228
0.085	3.3	0.4	560	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2,193.4	5,220	9,228
0.0875	2.9	0.4	518	[illegible]	0.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-66.22	[illegible]	2,149.1	5,250	9,228
0.090	3.0	0.4	524	[illegible]	0.3	1,550	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]			2,063.4		
0.0925	2.7	0.3	472	[illegible]	0.3	1,450			[illegible]										
0.095	2.6	0.3	455	[illegible]	0.3	1,345													
0.0975																			
0.100																			

* Denotes maximum resistance or maximum displacement value.

used as a work sheet in conjunction with the dynamic analysis in table 9.39, indicates the moment resistance developed in the shear wall members at each time increment. These moment values are used to determine the point in the dynamic analysis (table 9.39) when the shear is exceeded and at which the resistance functions will change. The terms used as column headings are discussed below:

x_2 = displacement of mass m_2 from table 9.39.

x_3 = displacement of mass m_3 from table 9.39.

For interior walls:

M_{AB} = moment at end of member AB. Limiting value of moment is obtained by determining the moment existing at the end of the member when the shear stress is 600 psi. A shear stress of this magnitude requires about one percent of web reinforcement (eq 4.24a) which is a practical upper limit for shear reinforcement.

If v = 600 psi:

$$V = vbjd = \frac{(600)}{1{,}000}\, 10\left(\frac{7}{8}\right) 45 = 236 \text{ kips}$$

$$V = \frac{2 \text{ moment}}{\text{span}} = 236 = \frac{2M}{30.44}$$

$$M = 3{,}600 \text{ ft-kips}$$

The moment existing at any time t is obtained from the moment distributions in paragraphs 9-25m and 9-25n. For a unit deflection, $M_{AB} = 1.0(10)^6 x_2 - 0.67(10)^6 x_3$.

M_{CD} = moment at end of member CD. For limiting elastic value, where v = 600 psi (practical upper limit)

$$V = vbjd = \frac{(600)}{1{,}000}\, 10\left(\frac{7}{8}\right) 45 = 236 \text{ kips}$$

$$V = \frac{2 \text{ moment}}{\text{span}} = 236 \quad \frac{2M}{31.37}$$

$$M = 3{,}700 \text{ ft-kips}$$

The moment existing at any time t obtained from moment distributions (par. 9-25m and 9-25n) is, for a unit deflection,

$$M_{CD} = 0.49(10)^6\, x_3 + 0.06(10)^6\, x_2 \ .$$

M_{EF} = moment at end of member EF. For limiting elastic value, where v = 600 psi

$$V = vbjd = (600)\, 30\left(\frac{7}{8}\right) 33 = 519 \text{ kips}$$

$$V = \frac{2\text{ moment}}{\text{span}} = 519 = \frac{2M}{32.3}$$

$M = 8,360$ ft-kips

The moment existing at any time t obtained from moment distributions (par. 9-25m and 9-25n) is for a unit deflection,

$M_{EF} = 0.22\ x_3 + 0.75\ x_2$.

R_3 = resistance acting upon mass m_3 obtained from paragraph 9-25o where, for one interior wall, $R_3 = 5.09(10)^5\ x_3 - 7.06(10)^5\ x_2$. The resistance acting upon mass m_3 is also equal to the resistance developed in the second-story shear wall. When first cracking occurs, $f_c = 0.1f'_{dc} = 390$ psi (eq 4.54).

$V = vbd = (390)10(20.5)12 = 960$ kips

Thus, for two wall elements AC and BD, the limiting value is $2(960) = 1,920$ kips

R_2 = resistance acting upon mass m_2 obtained from paragraph 9-25o where, for one interior shear wall, $R_2 = 14.36(10)^5\ x_2 - 7.22(10)^5\ x_3$.

$\frac{R_2 + R_3}{2}$ = resistance developed in first-story shear wall element, CE or DF. When cracking occurs $v = 390$ psi. Hence, limiting value equals $V = vbd = (390)10(20.5)12 = 960$ kips

For end walls:

R_3 = resistance acting upon mass m_3 obtained from paragraph 9-25x where, for the exterior wall, $R_3 = \left(\frac{1}{2}\right)\left[7.16(10)^5\ x_3 - 7.16(10)^5\ x_2\right]$. The resistance acting upon the mass m_3 is also equal to the resistance developed in the second-story shear wall. When cracking occurs, $v = 390$ psi, limiting value,

$V = vbd = (390)10(48)12 = 2,250$ kips

R_2 = resistance acting upon mass m_2 obtained from paragraph 9-25x where, for one end shear wall, $R_2 = \left(\frac{1}{2}\right)\left[13.71(10)^5 x_2 - 7.16(10)^5 x_3\right]$.

$\frac{R_2 + R_3}{2}$ = resistances developed in first-story shear wall element, CE or DF. When cracking occurs, $v = 390$ psi; hence, limiting value equals $V = vbd = (390)10(20.5)12 = 960$ kips

M_{EF} = moment at end of member EF. For limiting elastic value, $v = 600$ psi (practical upper limit)

M = 8,360 ft-kips (same as M_{EF} for interior wall above since dimensions are the same).

The moment existing at any time t obtained from the moment distribution (par. 9-25u) is $M_{EF} = \left(\frac{1}{2}\right) 7.6(10)^5 \ x_2$.

From table 9.40 the following resistance modifications were made. At time, t = 0.025 the shear in member AB of the interior wall is exceeded as indicated by moment values (3,788 > 3,600); however, the change in resistance of the interior wall is not made until the shear is exceeded in all portal members or a wall member. At time t = 0.0250 the shear stress in member CD is exceeded as indicated by moment values (4,370 > 3,700) and the shear in the lower wall is greater than first cracking value (1,020 > 960). The resistance of the lower shear wall is considered constant (at

Table 9.40. Criteria for Resistance Modification

			Interior Walls						End Walls			
									AC		CE	
t (sec)	x_2 (10^{-5} ft)	x_3 (10^{-5} ft)	M_{AB} (ft-kips)	M_{CD} (ft-kips)	M_{EF} (ft-kips)	R_3 (kips)	R_2 (kips)	$\frac{R_3}{2} + \frac{R_2}{2}$ (kips)	R_3 (kips)	R_2 (kips)	$\frac{R_3}{2} + \frac{R_2}{2}$ (kips)	M_{EF} (ft-kips)
Maximum allowable value - -			3,600	3,700	8,360	1,920		960	[illegible]		960	8,360
0	0	0	0									
0.0025	6.80	-5.50	-100.6									
0.005	19.46	-30.4	-434									
0.0075	33.20	-29.6	-518									
0.010	57.37	+30.9	-25									
0.0125	99.74	130.7	639									
0.015	171.11	253.1	1,381									
0.0175	256.72	386.1	2,136									
0.020	360.48	529.1	2,771				[illegible]					
0.0225	450.14	680.8	3,778*	3,611	4,875	290	1,570	930				1,710
0.025	520.0	830.0	4,820	4,372*	5,730	550	1,400	[illegible]*	1,110	[illegible]	852	1,970
0.0275	577.2	953.2	(5,662)			550		(1,110)	1,35[illegible]	[illegible]	945	2,190
0.030	626.57	1,048.0	(6,280)			550		(1,165)	1,5[illegible]	550	1,030*	2,380
0.0325	673.20	1,108.4	(6,564)	(5,036)		550		([illegible])	1,9[illegible]0	(655)	(1,108)	2,560
0.035	723.6	1,153.6	(6,686)			550			1,540			2,750
0.0375	787.8	1,176.2	(6,492)			550	(2,830)	(1,530)	1,390	([illegible])	(1,290)	2,980
0.040	857.4	1,159.6	(5,876)	(6,194)		550	(3,[illegible])	([illegible])	1,090	([illegible])	(1,402)	3,250
0.0425	944.76	1,140.8	(5,068)			550			(704)	(2,375)	(1,539)	3,580
0.045	1,006.8	1,138.9	(4,229)	(6,210)		550	([illegible])	(2,450)	475	([illegible])	(1,667)	3,820
0.0475	1,027.5	1,164.1	(4,741)	(6,338)		550	(6,400)	(2,535)	450	([illegible])	(1,670)	3,900
0.050	1,013.7	1,218.1	(5,301)	(6,560)		550	(5,580)	2,355*	(735)	(2,515)	1,625*	3,840
0.0525	1,003.0	1,279.9	(6,079)	(6,860)		-900	5,180	[illegible]	560	[illegible]	1,645	3,800
0.055	1,026.3	1,329.6	(6,426)	(7,115)		-830	5,200	2,350	[illegible]	2,275	1,683	3,900
0.0575	1,093.8	1,345.2	(6,102)	(7,255)		-1,200	6,000	2,565*	[illegible]	2,580	1,787*	4,150
0.060	1,200.5	1,332.8	(5,278)	(7,270)		(-1;710)	(7,630)	2,[illegible]0	473	(3,475)	(1,974)	4,560
0.0625	1,335.1	1,304.9	(4,099)	(7,200)		(-2,770)	(9,780)	(3,505)	-108	(4,475)	(2,184)	5,070
0.065	1,481.3	1,371.4	(3,794)	(7,610)		(-3,580)	(11,400)	(3,[illegible])	-3[illegible]4	([illegible])	(2,446)	5,630
0.0675	1,629.0	1,443.9	(3,539)	(8,036)		(-4,150)	(12,950)	(4,400)	-[illegible]65	(6,040)	(2,668)	6,180
0.070	1,764.6	1,528.3	(3,483)	(8,560)		(-4,690)	(14,300)	(4,805)	-846	([illegible])	(2,889)	6,700
0.0725	1,887.2	1,624.7	(3,597)	(9,090)		(-5,090)	(15,350)	(5,130)	-[illegible]42	(7,140)	(3,099)	7,170
0.075	1,991.1	1,730.4	(3,954)	(9,660)		(-5,280)	16,100	(5,410)	-534	(7,450)	(3,258)	7,560
0.0775	2,074.2	1,838.5	(7,995)	(10,340)		(-5,290)	16,500	(5,665)	-542	(7,675)	(3,417)	7,860
0.080	2,135.2	1,938.7	(5,087)	(10,780)		(-5,250)	16,750	(5,750)	-934	(7,700)	(3,383)	8,100
0.0825	2,172.0	2,019.2	(5,642)	(11,170)		(-5,400)	16,700	(5,650)	-548	(7,675)	(3,563)	8,250
0.085	2,195.1	2,066.8	(5,968)	(11,465)		(-5,000)	16,450	(5,725)	-460	(7,650)	(3,595)	8,340
0.0875	2,193.4	2,071.9	(6,019)	(11,465)		(-4,960)	15,580	(5,310)	-436	(7,600)	(3,582)	6,350
0.090	2,150.8	2,025.0	(5,850)	(11,240)		(-4,890)	16,250	(5,680)	-450	(7,500)	(3,525)	8,190
0.0925	2,066.8	1,925.5	(5,405)	(10,682)								

* A change in the resistance functions occurs at this step.
( ) Indicate use of elastic resistance relationship.

first cracking value of 960 kips) as the wall deforms into the plastic range. New resistances are computed in paragraph 9-26e.

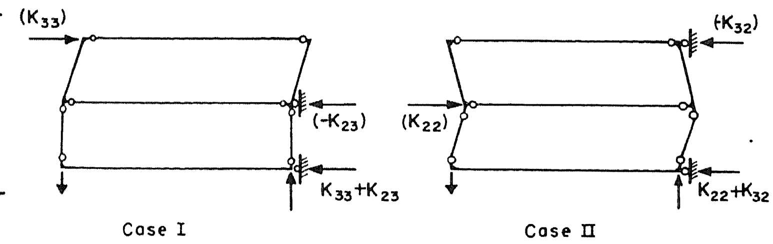
At time t = 0.030 the shear in the lower end wall is greater than first cracking value (1,030 > 960). New resistance values are computed in paragraph 9-26f. After an element is considered to be in the plastic range, the possibility of reversal to the elastic range still exists at which time a new resistance function must be obtained. In order to determine when this return to the elastic range exists, a value of shear is computed using the original elastic equations and is placed in parentheses in table 9.40. If this elastic value at time $t = t_{n+1}$ is less than the elastic value at time $t = t_n$, a reversal occurs and new resistance functions are computed.

At time t = 0.0500 the first-story shear walls (members CE) return to the elastic range (2,355 < 2,535 and 1,625 < 1,670), hence, new resistance functions are computed in paragraph 9-26i.

At time t = 0.0575 the first-story shear walls have become plastic again, hence, require revised resistance functions (par. 9-26f).

From time t = 0.0575 to the end of the analysis the same resistance function is used; however, member AB in the interior shear wall indicates a reversal to the elastic range at time t = 0.060 and at time t = 0.090. These portal members are considered to be effective in a change in resistance function only if three members (AB, CD, and EF) change to the elastic range.

e. Resistance Function (t = 0.0250) for Table 9.39. At t = 0.0250 the shear capacity in members AB, CD, and the first-story shear wall CE is exceeded, hence, new resistance factors must be computed for the interior shear wall.



Since the interior shear walls will offer no further resistance after t = 0.025, the resistance for the entire building will be computed as follows:

For $x_2 = 520.0(10)^{-5}$ ft; $x_3 = 830.0(10)^{-5}$ ft

$$R_3 = 27.52(10)^5\ x_3 - 35.4(10)^5\ x_2$$
$$= (27.52)830 - (35.4)520 = 22,800 - 18,400 = 4,400$$

For $t < 0.0250$

$$R_3 = 4,400 + 7.16(10)^5\left[x_3 - 830.0(10)^{-5}\right] - 7.16(10)^5\left[x_2 - 520(10)^{-5}\right]$$
$$= 4,400 + 7.16(10)^5\ x_3 - 5,940 - 7.16(10)^5\ x_2 + 3,720$$
$$= 2,180 + 7.16(10)^5\ x_3 - 7.16(10)^5\ x_2$$

At $t = 0.0250$

$$R_2 = 71.15(10)^5\ x_2 - 36.01(10)^5\ x_3$$
$$= 71.15(10)^5\ 520(10)^{-5} - 36.01(10)^5\ 830(10)^{-5}$$
$$= 7,100$$

For $t > 0.025$

$$R_2 = 7,100 + 13.71(10)^5\left[x_2 - 520(10)^{-5}\right] - 7.16(10)^5\left[x_3 - 830(10)^{-5}\right]$$
$$= 7,100 + 13.71(10)^5\ x_2 - 7,140 - 7.16(10)^5\ x_3 + 5,940$$
$$= 5,900 + 13.71(10)^5\ x_2 - 7.16(10)^5\ x_3$$

f. <u>Resistance Function (t = 0.030) for Table 9.39.</u> At t = 0.030 the first-story end shear wall becomes plastic, hence requires new resistance function for entire building.

End wall resistance:

$$R_3 = K_{33}\ \delta_3 + K_{32}\ \delta_2 = 716,000\ x_3 - 716,000\ x_2$$
$$R_2 = K_{22}\ \delta_2 + K_{23}\ \delta_3 = 716,000\ x_2 - 716,000\ x_3$$

For entire building:

At $t = 0.0300$

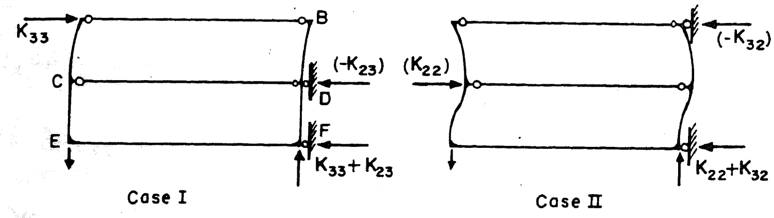
$$R_3 = 2,180 + (7.16)1,048 - 7.16(626.57) = 5,200$$
$$R_2 = 5,900 + (13.71)626.57 - (7.16)1,048.0 = 7,000$$

At $t > 0.030$

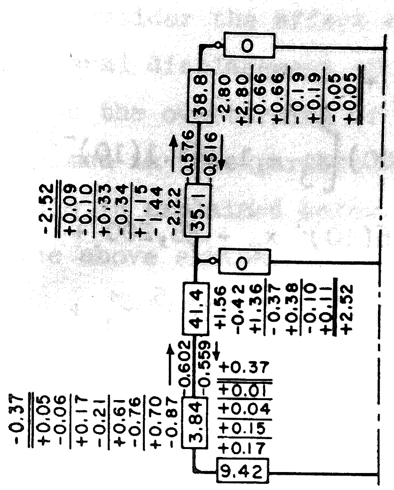
$$R_3 = 5,200 + 7.16(10)^5\left[x_3 - 1,048(10)^{-5}\right] - 7.16(10)^5\left[x_2 - 626.57(10)^5\right]$$
$$= 5,200 + 7.16(10)^5\ x_3 - 7,520 - 7.16(10)^5\ x_2 + 4,480$$
$$= 2,160 + 7.16(10)^5\ x_3 - 7.16(10)^5\ x_2$$
$$R_2 = 7,000 + 7.16(10)^5\left[x_2 - 626.57(10)^{-5}\right] - 7.16(10)^5\left[x_3 - 1,048(10)^{-5}\right]$$
$$= 7,000 + 7.16(10)^5\ x_2 - 4,480 - 7.16(10)^5\ x_3 + 7,500$$
$$= 10,020 + 7.16(10)^5\ x_2 - 7.16(10)^5\ x_3$$

g. <u>Resistance Function (t = 0.0500) for Table 9.39.</u> At t = 0.500 the elastic resistance check (table 9.40) shows that the stress in the

lower shear wall elements in both interior and exterior walls have reversed so that the resistance functions must be recalculated, taking into account the new elastic resistance of these elements.



h. Moment Distribution - Case I (t = 0.0500).



i. Moment Distribution - Case II (t = 0.0500).

From case I distribution:

$$K_{33} = (2)2.52(10)^6/11.6 = 434,000$$

$$K_{23} = 2\left[2.52(10)^6 + (2.52 - 0.37)(10)^6\right]/11.6$$

$$= 806,000$$

From case II distribution:

$$K_{22} = 2\left[3.73(10)^6 + (3.73 + 0.45)(10)^6\right]/11.6$$

$$= 1,366,000$$

$$-K_{32} = (2)3.73(10)^6/11.6 = 644,000$$

Resistance function for one interior shear wall (members AB and CD plastic)

$$R_3 = K_{33}\ x_3 + K_{32}\ x_2 = 434,000\ x_3 - 644,000\ x_2$$

$$R_2 = K_{22}\ x_2 + K_{23}\ x_3 = 1,366,000\ x_2 - 806,000\ x_3$$

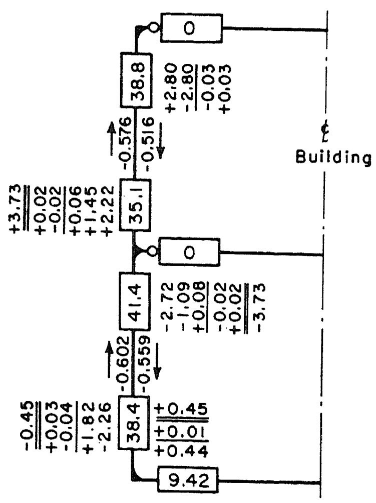
At t = 0.0475, using previous resistance functions for entire building

$$R_3 = 2,160 + 7.16(10)^5\ x_3 - 7.16(10)^5\ x_2$$

$$= 2,160 + (7.16)(1,164.1) - (7.16)(1,027.5) = 3,136$$

$$R_2 = 10,020 + 7.16(10)^5\ x_2 - 7.16(10)^5\ x_3$$

$$= 9,040$$



For entire building, resistances for times less than t = 0.0475 will be calculated using the original elastic expressions for the end wall and the new expression calculated for plastic members AB and CD above.

$$R_3 = 3,136 + (4)433,000\left[x_3 - 1,164.1(10)^{-5}\right] +$$

$$\frac{2}{2}(716,000)\left[x_3 - 1,164.1(10)^{-5}\right] - (4)644,000\left[x_2 - 1,027.5(10)^{-5}\right] -$$

$$\frac{2}{2}(716,000)\left[x_2 - 1,027.5(10)^{-5}\right] = 3,136 + 17.3(10)^5\ x_3 - 20,200 +$$

$$7.16(10)^5\ x_3 - 8,350 - 25.7(10)^5\ x_2 + 26,400 - 7.16(10)^5\ x_2 + 7,360$$

$$R_3 = 8,346 + 24.46(10)^5\ x_3 - 32.86(10)^5\ x_2$$

$$R_2 = 9,040 + (4)1,366,000\left[x_2 - 1,027.5(10)^{-5}\right] +$$

$$\frac{2}{2}(1,371,000)\left[x_2 - 1,027.5(10)^{-5}\right] - (4)(806,000)\left[x_3 - 1,164.1(10)^{-5}\right] -$$

$$\frac{2}{2}(716,000)\left[x_3 - 1,164.1(10)^{5}\right] = 9,040 + 54.6(10)^5\ x_2 - 56,200 +$$

$$13.71(10)^5\ x_2 - 14,100 - 32.3(10)^5\ x_3 + 37,600 - 7.16(10)^5\ x_3 +$$

$$8,350 = -15,310 + 68.31(10)^5\ x_2 - 39.46(10)^5\ x_3$$

j. Resistance Functions ($t = 0.0575$) for Table 9.39. At $t = 0.0575$ the shear wall deflections are of such magnitude that the lower wall elements are again in the plastic range and new resistance functions must be found for the entire building.

At $t = 0.0550$

$$R_3 = 8,346 + 24.46(10)^5\ x_3 - 32.86(10)^5\ x_2 = 8,346 + 24.46(1,329.6) - (32.86)1,026.3 = 7,046$$

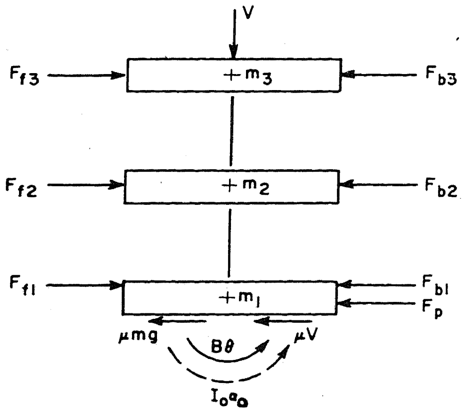
$$R_2 = -15,310 + 68.31(10)^5\ x_2 - 39.46(10)^5\ x_3 = -15,310 + 68.31(1,026.3) - (39.46)1,329.6 = 2,190$$

For $t > 0.0550$ (all members plastic except second story of end wall)

$$R_3 = 7,046 + 7.16(10)^5\left[x_3 - 1,329.6(10)^{-5}\right] - 7.16(10)^5\left[x_2 - 1,026.3(10)^{-5}\right] = 7,046 + 7.16(10)^5\ x_3 - 9,520 - 7.16(10)^5\ x_2 + 7,360 = 4,876 + 7.16(10)^5\ x_3 - 7.16(10)^5\ x_2$$

$$R_2 = 2,190 + 7.16(10)^5\left[x_2 - 1,026.3(10)^{-5}\right] - 7.16(10)^5\left[x_3 - 1,329.6(10)^{-5}\right] = 2,190 + 7.16(10)^5\ x_2 - 7,360 - 7.16(10)^5\ x_3 + 9,520 = 4,350 + 7.16(10)^5\ x_2 - 7.16(10)^5\ x_3$$

9-27 DYNAMIC OVERTURNING AND SLIDING ANALYSIS. a. Dynamic Analysis. The rigid body overturning and sliding analysis performed in paragraph 9-24e



is necessary in order to obtain the accelerations of various portions of the structure. These accelerations are used in the preliminary investigation of the shear walls (par. 9-25). A further refinement in the design would include a dynamic overturning and sliding analysis (par. 9-06c) in order to consider the effect of the relative lateral displacement of masses m_2 and m_3 upon the overturning of the structure. This dynamic analysis is not performed in this particular example; however, the procedure for performing this is explained below. The forces acting on the structure are shown in the above sketch.

The masses and centroid locations can be obtained from paragraph 9-26a. The dynamic analysis can be performed by a concurrent numerical integration of equations (9.8), (9.9), and (9.10). When sliding does not occur, equation (9.11) will be used in place of equation (9.8).

$$\alpha_o = \frac{M_o - (F_3 - R_3)y_3 - (F_2 - R_2)y_2 - (F_1 - R_1)y_1}{I_o - m_3y_3^2 - m_2y_2^2 - m_1y_1^2} \quad \text{(eq 9.8)}$$

$$\ddot{x}_o = \frac{F_1 - R_1}{m_1} - \alpha_o y_1 \quad \text{(eq 9.9)}$$

$$\ddot{x}_2 = \frac{F_2 - R_2}{m_2} - (\ddot{x}_o + \alpha_o y_2) \quad \text{(eq 9.10)}$$

$$\ddot{x}_3 = \frac{F_3 - R_3}{m_3} - (\ddot{x}_o + \alpha_o y_3) \quad \text{(eq 9.10)}$$

$$\alpha_o = \frac{M_o - (F_3 - R_3)y_3 - (F_2 - R_2)y_2}{I_o - m_3y_3^2 - m_2y_2^2} \quad \text{(eq 9.11)}$$

The description and location of terms in this example to be used for the analysis are described below:

M_o = moment of all external forces about axis of rotation

$= (F_{f3} - F_{b3})y_{f3} + (F_{f2} - F_{b2})y_{f2} + (F_{f1} - F_{b1})y_{f1} - F_p y_p - B\theta$

where

F_{f3} = front wall slab reaction at roof level (table 9.29)

F_{b3} = back wall slab reaction at roof level(table 9.30)

y_{f3} = vertical distance from axis of rotation to location of wall slab reactions at second floor level

y_{f2} = vertical distances from axis of rotation to location of wall slab reactions at second floor level

y_{f1} = vertical distance from axis of rotation to location of wall slab reactions at first floor level

F_{f2} = front wall slab reactions at second floor level (table 9.29)

F_{b2} = back wall slab reactions at second floor level (table 9.30)

F_{f1} = front wall slab reactions at first floor level (table 9.29)

F_p = passive resistance of earth (par. 9-24c)

$B\theta$ = moment of earth resistance due to overturning (par. 9.24b)

F_3 = summation of external horizontal forces applied to the structure at mass $m_3 = (F_{f3} - F_{b3})$

R_1, R_2, R_3 = internal shear wall resistance acting on mass m_1, m_2, m_3 (table 9.39)

y_1, y_2, y_3 = distance from centroid of mass m_1, m_2, and m_3 to axis of rotation.

F_2 = summation of external horizontal forces applied to the structure at mass $m_2 = (F_{f2} - F_{b2})$

F_1 = summation of external forces on mass $m_1 = F_{f1} - F_{b1} - F_o - \mu mg - \mu V$

I_o = polar moment of inertia (table 9.13a)

$\ddot{x}_o$ = horizontal acceleration of axis of rotation "o"

$\ddot{x}_2$ = horizontal acceleration of mass m_2

$\ddot{x}_3$ = horizontal acceleration of mass m_3

Using the acceleration impulse extrapolation method (eq 5.49), values for θ, x_o, x_2, and x_3 can be obtained by the consecutive application of the following numerical equations:

$$(\theta)_{t = n + 1} = 2(\theta)_{t = n} - (\theta)_{t = n - 1} + (\alpha_o)_{t = n} (\Delta t)^2$$

$$(x_o)_{t = n + 1} = 2(x_o)_{t = n} - (x_o)_{t = n - 1} + (\ddot{x}_o)_{t = n} (\Delta t)^2$$

$$(x_2)_{t=n+1} = 2(x_2)_{t=n} - (x_2)_{t=n-1} + (\ddot{x}_2)_{t=n} (\Delta t)^2$$
$$(x_3)_{t=n+1} = 2(x_3)_{t=n} - (x_3)_{t=n-1} + (\ddot{x}_3)_{t=n} (\Delta t)^2$$

The rotation of the building θ can then be used for investigation of soil pressure due to overturning, and values of x_2 and x_3 can be used to determine shear wall resistances which are developed during the overturning and sliding action.

b. Soil Pressure Investigation. In order to determine maximum and minimum soil pressures at front and rear wall footings it will be necessary to summarize the dynamic reactions on these elements. Since a portion of the building has been considered to offer no resistance to overturning it will be necessary to consider only those footings of the portion of the building which rotates.

Area of footings of rotating portion of building:

Front and rear wall footings	= 2(2.5)158.5 =	792 ft^2
Shear and end wall footings	= 6(2.5)44.75 =	672 ft^2
Attached column footings	= 20(2.67)1.67 =	89 ft^2
	Total area =	1,553 ft^2

c. Dead Load Soil Pressure. Mass of overturning elements of building is obtained from table 9.32. This is tabulated below:

Front wall	= 15.05 kip-sec^2/ft
Rear wall	= 15.05
End walls	= 8.50
Shear walls	= 16.05
Roof slab	= 12.12
Transverse roof girders	= 0.95
Longitudinal roof girders	= 0.56
Second floor columns	= 0.94
Second floor slab	= 9.90
Transverse second floor girders	= 0.71
Longitudinal second floor girders	= 0.57
First floor columns	= 1.12
Attached footings	= 0.34
Front and rear footings	= 3.08
Shear and end wall footings	= 7.75
Occupancy load - second floor	= 2.15
Occupancy load - first floor	= 2.15
Total mass	= 87.99 kip-sec^2/ft

$W = mg = (87.99)32.2 = 2{,}830$ kips

To the above tabulation will be added the weight of soil and slab directly above that portion of the footings which participate in the rotation.

	Dimensions (ft)		Volume (cu ft)		Weight (kips)
Earth above shear wall footings	= 4(1.67)1.67(46.0)	=	512	=	51.2
Earth above end wall footings	= 2(0.83)1.67(46.0)	=	127	=	12.7
Earth above front and rear wall footings	= 2(0.83)1.67(156.0)	=	432	=	43.2
Slab above shear wall footings	= 4(1.67)0.5(46.0)	=	154	=	23.1
Slab above end wall footings	= 2(0.83)0.5(46.0)	=	38	=	5.7
Slab above front and rear wall footings	= 2(0.83)0.5(156.0)	=	129	=	19.4
					155.3

Total dead load = 2,830 + 155 = 2,985 kips

Average dead load soil pressure $= \frac{2,985}{1,553} = 1.920$ kips/ft^2

d. Blast Load Soil Pressure. The blast load soil pressures will consist of a pressure due to vertical blast loads and a pressure due to rotation of the structure. It is necessary to determine the vertical blast load on that portion of the building which is considered to rotate. This will be obtained by subtracting the blast load on the portion of the structure not rotating from the total blast load.

Portion of Total Vertical Blast Loading Acting upon Non-rotating Part of Structure	Net Vertical Blast Load upon Rotating Part of Structure
16 "a" edge slab reactions	$52 - 16 = 36\ V_A$
16 "b" edge slab reactions	$52 - 16 = 36\ V_B$
8 bays of corridor slab plus blast load directly on girders $= \frac{872(144)\overline{P}_{roof}}{1,000} = 125\ \overline{P}_{roof}$	$307 - 125 = 182\ \overline{P}_{roof}$

Soil pressure due to overturning $= \frac{\theta Bc}{I}$

c = distance from axis of rotation to center of front or rear wall footing = 23.7 ft

$$I = 6\left(\frac{1}{12}\right)2.5(44.75)^3 + 2\left[\frac{158.5(2.5)^3 + 396(23.6)^2}{12}\right] = 552,165\ \text{ft}^4$$

Overturning pressure $= \frac{23.7\ \theta B}{552,165} = 4.30(10)^{-5}\ \theta B$

e. Summary. These soil pressures are summarized in table 9.41. Maximum pressure (occurring at t = 0.0525) = 22.31 kips/ft^2. This indicates satisfactory footing design since 22.31 < 30.00 kips/ft^2. A negative pressure or tension develops on the opposite side of the building. This is considered to be zero pressure when analyzing the walls as deep beams.

Table 9.41. Summary of Earth Pressures

The maximum soil pressure consists of a summation of pressure due to loads V plus dead load plus stresses from moment θB.
The minimum soil pressure consists of a summation of pressure due to loads V plus dead load minus stresses from moment θB.

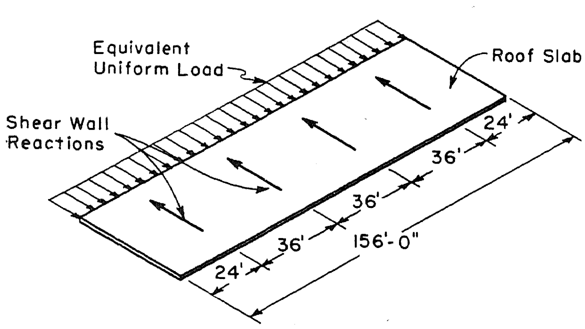
t (sec)	V_A (kips)	$36V_A$ (kips)	V_B (kips)	$36V_B$ (kips)	$\bar{P}_{roof}$ (psi)	V_c* (kips)	V** (kips)	θB (ft-kips)	Dead Load Soil Pressure (kips/ft^2)	Blast Load Soil Pressure† (kips/ft^2)	Overturning Soil Pressure (kips/ft^2)	Minimum Soil Pressure (kips/ft^2)	Maximum Soil Pressure (kips/ft^2)
0	0	0	0	0	0	0	0	0	1.92	0	0	1.92	1.92
0.0025	1.12	40.4	2.48	89.2	0.62	113	243	1,325		0.08	0.05	0.95	2.05
0.005	3.03	109.0	6.51	234.0	1.25	229	572	5,340		0.34	0.23	2.37	2.83
0.0075	5.86	211	13.10	472.0	1.90	346	1,029	12,000		0.68	0.52	2.08	3.12
0.010	9.69	348	22.22	798	2.55	465	1,611	21,200		1.03	0.91	2.04	3.86
0.0125	14.12	507	32.80	1,180	3.18	579	2,266	32,700		1.46	1.41	1.97	4.79
0.015	18.58	668	43.20	1,555	3.80	692	2,915	46,300		1.88	1.99	1.81	5.79
0.0175	22.46	810	53.00	1,910	4.45	810	3,530	62,000		2.27	2.67	1.52	6.86
0.020	25.43	916	59.90	2,160	5.10	930	4,006	79,200		2.61	3.40	1.13	7.93
0.0225	27.35	984	64.20	2,310	5.72	1,040	4,334	98,000		2.79	4.22	0.49	8.93
0.025	28.55	1,030	66.70	2,400	6.35	1,155	4,585	118,000		2.93	5.08	-0.23	9.93
0.0275	29.82	1,075	69.30	2,440	7.00	1,275	4,790	138,000		3.08	5.94	-0.94	10.94
0.030	31.50	1,130	73.00	2,500	7.62	1,388	5,018	176,000		3.23	7.57	-2.42	12.72
0.0325	34.30	1,235	79.60	2,870	8.30	1,510	5,615	213,000		3.61	9.16	-3.63	14.69
0.035	37.55	1,350	87.50	3,150	8.60	1,565	6,065	248,000		3.90	10.65	-4.83	16.47
0.0375	39.10	1,410	91.20	3,280	8.50	1,548	6,238	282,000		4.01	12.10	-6.17	18.03
0.040	40.00	1,440	93.20	3,360	8.45	1,530	6,330	313,000		4.07	13.45	-7.46	19.44
0.0425	40.30	1,450	94.4	3,400	8.30	1,510	6,360	338,000		4.10	14.55	-8.53	20.57
0.045	40.50	1,460	94.8	3,410	8.20	1,490	6,360	357,000		4.10	15.35	-9.33	21.37
0.0475	39.3	1,415	92.8	3,340	8.10	1,478	6,390	372,000		4.10	16.00	-9.98	22.0
0.050	37.9	1,365	88.2	3,170	8.05	1,465	6,000	384,000		3.86	16.50	-10.72	22.28
0.0525	35.8	1,290	83.5	3,000	8.00	1,456	5,746	388,000		3.69	16.70	-11.09	22.31
0.055	35.4	1,275	82.6	2,980	7.92	1,440	5,695	388,000		3.66	16.70	-11.12	22.28
0.0575	35.0	1,260	81.5	2,940	7.82	1,422	5,622	383,000		3.62	16.45	-10.91	21.99
0.060	34.7	1,250	81.0	2,920	7.75	1,410	5,580	372,000		3.59	16.00	-10.49	21.51
0.0625	34.2	1,230	79.8	2,870	7.65	1,390	5,490	355,000		3.53	15.25	-9.80	20.70
0.065	33.8	1,218	78.6	2,830	7.55	1,375	5,423	335,000		3.49	14.40	-8.99	19.85
0.0675								310,000			13.30		
0.070								280,000			12.00		

* $V_C = 182\ \bar{P}_{roof}$ (fig. 9.29).

** $V = 36V_A + 36V_B + V_C$ = total vertical blast load.

† Blast load soil pressure $= \frac{P}{A} = \frac{V(\text{kips})}{1{,}553(\text{ft}^2)}$

9-28 ROOF AND FLOOR SLAB DESIGN (Deep Beam Action). a. Roof Slab Analysis. Using the resistance values R_3 obtained from the preliminary shear wall investigation (table 9.39) as concentrated loads applied to the roof slab acting as deep beam it is possible to obtain the equivalent uniform load acting on the slab required to develop this resistance.



b. Loads on Roof Slab.

Maximum R_3 = 7,046 kips (table 9.39, t = 0.0550 sec)

Uniform load $= \frac{7,046}{156} = 45.2$ kips/ft

The distribution of this uniform load to the roof slab will be determined by proportioning the load in accordance with relative resistance offered by end and interior shear walls.

In order to determine relative resistance of end and interior shear walls it will be necessary to first determine resistance of the end walls.

The resistance of the end wall at the time of the maximum total resistance is:

$R_3 = 7.16x_3 - 7.16x_2$ (par. 9-26d)

$x_3 = 1,329.6(10)^{-5}$ ft (table 9.39)

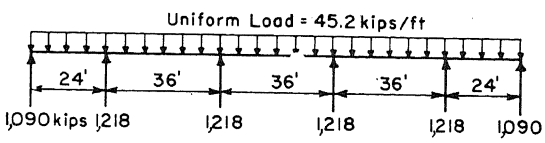
$x_2 = 1,026.3(10)^{-5}$ ft (table 9.39)

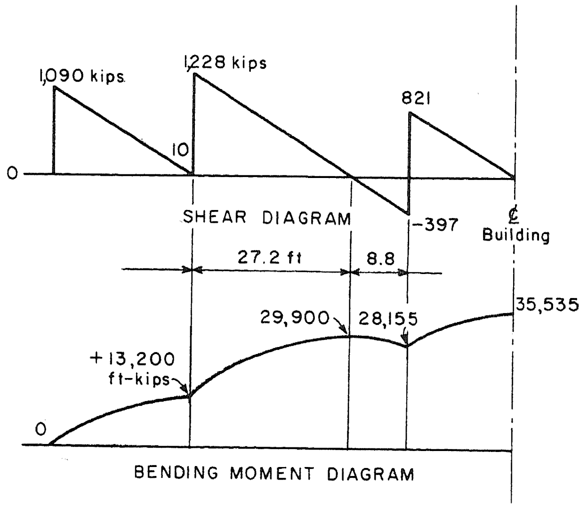
$\therefore R_3 = 2,175$ kips

Effective resistance for each end wall (par. 9-25y) $= \frac{1}{2} R_3 = \left(\frac{1}{2}\right) 2,175$ $= 1,090$ kips

Resistance for each interior wall $= \frac{7,046 - 2(1,090)}{4} = 1,218$ kips

Roof slab loading based upon relative shear wall resistances and corresponding reactions are shown below:





c. <u>Shear Strength.</u>

Shear check (roof):

$$v = \frac{\text{maximum shear}}{\text{area of roof section}}$$

$$= \frac{1,228(1,000)}{25.8(144)} = 330 \text{ psi}$$

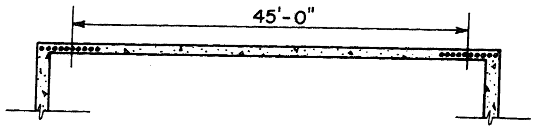
$330 < 0.10f'_{dc} = 390$ OK

Shear check (wall):

$$v = \frac{\text{maximum reaction}}{\text{area of wall section}} = \frac{1,228(1,000)}{0.83(48.0)144} = 215 \text{ psi}$$

$215 < 0.10f'_{dc} = 390$ OK

d. Steel Required for Bending. Maximum moment = 35,535 ft-kips (from moment diagram above). Assume this moment to be resisted entirely by steel placed within roof at front and back edges.



$A_s = \dfrac{M}{f_{dy}d} = \dfrac{35,535}{52.0(45.0)} = 15.20 \text{ in.}^2$ (eq 4.57)

Use 10 #11 bars, $A_s = 15.60 \text{ in.}^2$, $\Sigma o = 44.30$ in.

Bond investigation $= \dfrac{V}{\Sigma ojd} = \dfrac{1,228(1,000)}{44.3(7/8)\ 45(12)} = 58.7$ psi

$58.7 < 450 = 0.15f'_c \quad \therefore$ OK

e. Second Floor Slab Analysis. The second floor slab will be analyzed in a manner similar to that of the roof slab. Maximum $R_2 = 9,853$ kips (at t = 0.0450 in table 9.39).

f. Loads on Second Floor Slab. In determining individual reactions on basis of relative wall resistances, it is necessary to first determine resistance of end and interior shear walls. Since for R_2 both lower and upper portions of wall must be considered, it is necessary to consider changes in resistance as the various members become plastic.

End shear wall resistances:

For $0 < t \leq 0.030$ sec

$R_2 = 1,371,000\ x_2 - 716,000\ x_3$ (par. 9-25o)

At t = 0.030, $x_2 = 626.57(10)^{-5}$, $x_3 = 1,048.0(10)^{-5}$ (table 9.39)

$R_2 = 8,590 - 7,500 = 1,090$ kips (at t = 0.030)

For t > 0.030, at t = 0.0450, $x_2 = 1,006.8(10)^{-5}$, $x_3 = 1,138.9(10)^{-5}$

$R_2 = 1,090 + 716,000\ x_2 - 716,000\ x_3$ (par. 9-26f)

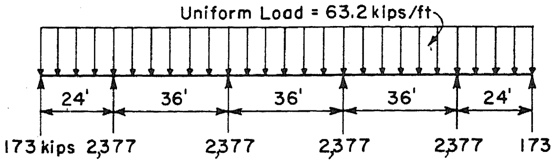
$= 1,090 - 7.16(132.1) = 345$ kips

Magnitude of total resistance offered by end and interior shear walls (in proportion to resistances).

Effective end wall, $R' = \dfrac{345}{2} = 173$ kips

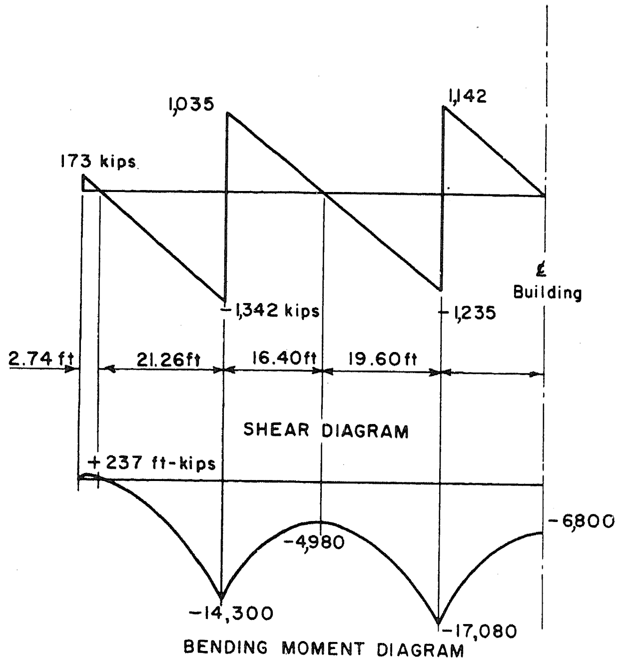
Interior wall, $R' = \dfrac{9,853 - 2(173)}{4} = 2,377$ kips

Second floor slab loading based upon relative shear wall resistances and corresponding reactions is shown below:



$$\text{Loading} = \frac{9,853 \text{ kips}}{156}$$

$$= 63.2 \text{ kips/ft}$$



g. Shear Strength.

Shear check (roof):

$$V = \frac{\text{maximum shear}}{\text{area of roof section}}$$

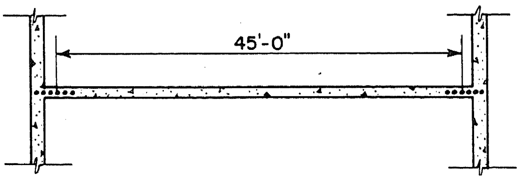
$$= \frac{1,342(1,000)}{25.8(144)} = 362 \text{ psi}$$

$362 < 0.10f'_{dc} = 390$ psi $\therefore$ OK

Shear check (wall):

$$V = \frac{\text{maximum reaction}}{2(\text{area of wall section})} = \frac{2,377(1,000)}{2(0.83)\ 48.0(144)} = 206 \text{ psi}$$

$206 < 0.10\ f'_{dc} = 390$ psi $\therefore$ OK



h. Steel Required for Bending.

Maximum moment = 17,080 ft-kips (from diagram in par. 9-28f)

Place steel in a manner similar to roof slab (par. 9-28d)

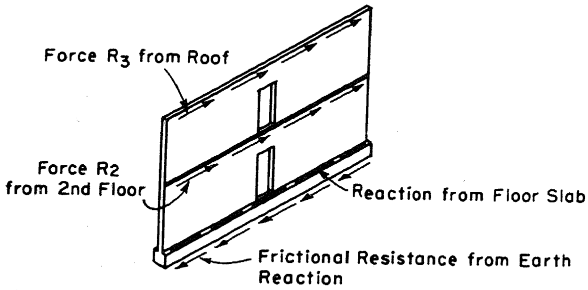
$$A_s = \frac{M}{f_{dy}d} = \frac{17,080}{52(45.0)} = 7.28 \text{ in.}^2 \quad (\text{eq } 4.57)$$

Use 6 #10 bars, $A_s = 7.62$ in.^{2}, $\Sigma o = 16.0$ in.

$$\text{Bond investigation} = u = \frac{V}{\Sigma ojd} = \frac{1,342(1,000)}{16.0\left(\frac{7}{8}\right)45(12)} = 178 \text{ psi}$$

$178 < 450 = 0.15f'_c$ $\therefore$ OK

i. First Floor Slab Analysis. Since the reaction from the shear wall deformation cannot be resisted completely by frictional forces under the shear wall and the earth pressures immediately behind the shear wall, it will be necessary to transmit the net force into the floor slab to be carried to the footings between the shear walls. In order to determine the magnitude of shears and moments in the floor slab, the reactions will be resisted by uniform loads on each span which are compatible with the reaction values for each span.



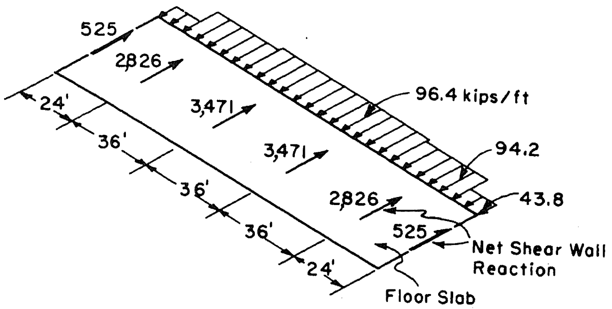
j. Loads on First Floor Slab. The net force acting on the floor slab is obtained by subtracting from the shear wall reactions the frictional force at time t = 0.0450 (in table 9.39, $R_2 + R_3$ is a maximum value at this time). Frictional force on base of shear wall at time t = 0.045 (see table 9.41) = μ(area)(soil pressure) = 0.75(2.5) 48.0(1.920 + 4.10) = 542 kips.

End shear wall reactions = (431.5 + 635.5) - 542 = 525 kips

First interior shear wall reactions = (1,417.3 + 1,951.3) - 542 = 2,826 kips

Center shear wall reactions = (1,677.2 + 2,336.2) - 542 = 3,471 kips

Assuming a distribution of uniform load for each half span on either side of each shear wall equal to the net force on the shear wall results in the following loading:



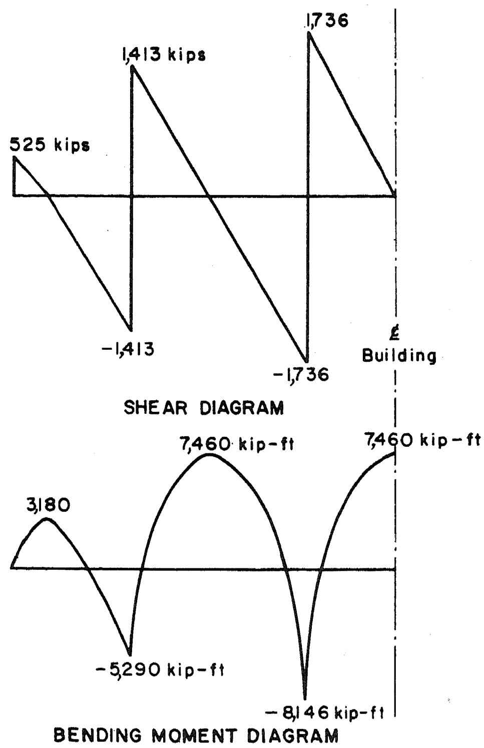
$$\frac{525}{(12)} = 43.8 \text{ kips/ft}$$

$$\frac{2,826}{\left(\frac{24 + 36}{2}\right)} = 94.2 \text{ kips/ft}$$

$$\frac{3,471}{\left(\frac{36 + 36}{2}\right)} = 96.4 \text{ kips/ft}$$

k. Steel Required for Bending.

$$A_s = \frac{M}{f_{dy} d} = \frac{8,146}{52(47.0)} = 3.33 \text{ in.}^2$$



Use 3 #10 bars, $A_s = 3.81 \text{ in.}^2$,

$\Sigma o = 12.0 \text{ in.}^2$

Bond requirement = required $\Sigma o = \frac{V}{ujd}$

$$= \frac{1,736(1,000)}{450\left(\frac{7}{8}\right)47(12)} = 7.82 \text{ in.} < 12.0 \therefore \text{ OK}$$

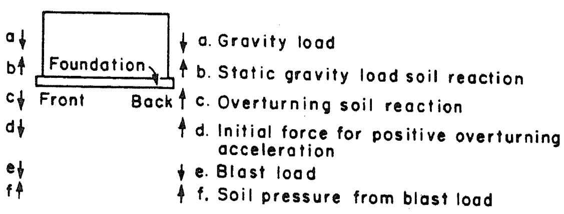
1. Shear Investigation. At the juncture of shear walls and floor slabs a maximum shear of 1,736 kips must be resisted. Since it is desirable to maintain integral construction of shear wall from top of footing to roof slab, it is necessary to design dowels to transmit this force across the construction joint.

Assuming the ratio of static yield stress in shear to static yield stress in tension of $\frac{21}{41.6}$ (par. 4-03d), the steel necessary per foot of slab,

$$A_s = \frac{1,736}{\left(\frac{21}{41.6}\right)52(46)} = 1.435 \text{ in.}^2/\text{ft}$$

Use #7 at 5 in. for dowels, $A_s = 1.44 \text{ in.}^2$

9-29 WALL ANALYSIS (Deep Beam Action). As a result of the overturning action of the structure there will be a shear force at the intersection of exterior wall and shear wall. In order to determine the time at which this shear force is a maximum, an analysis will be made in table 9.43. The shear forces and their direction acting on the wall are shown in the sketch at the left.



It is necessary to first compute the mass and centroid of wall section so that the inertial force from the overturning acceleration may be determined.

a. <u>Computation of Mass and Centroid Location of 24-ft Section of Exterior Wall.</u>

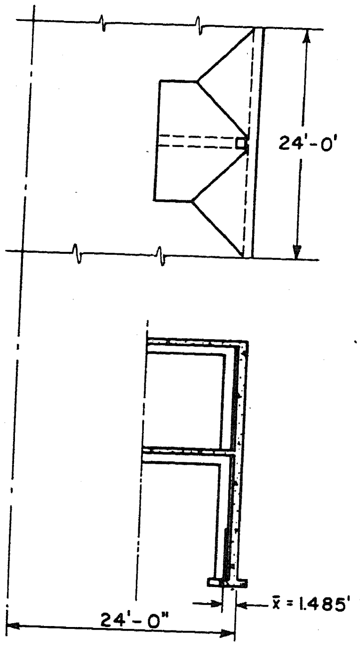
Table 9.42. Location of Centroid of Exterior Wall Section

Member	Dimension (ft)	Volume (ft^3)	Mass ($kip\text{-}sec^2/ft$)	$\bar{x}$* (ft)	$m\bar{x}$ ($kip\text{-}sec^2$)
Roof slab	$(0.54)12.0\left(\frac{19.17}{2}\right) + 0.54(12.0)3.0$	81.5	0.380	$\left(\frac{19.17}{4}\right)$	1.820
Second floor column	$1.0(1.0)10.0$	10.0	0.047	2.0	0.094
Wall	$0.83(25.17)24.0$	504	2.350	0.41	0.965
Roof girder	$1.33\left(\frac{19.17}{2}\right)0.96$	12.2	0.057	$\left(\frac{19.17}{4}\right)$	0.273
Second floor slab	$(0.46)12.0\left(\frac{19.17}{2}\right) + 0.46(12.0)3.0$	69.3	0.323	$\left(\frac{19.17}{4}\right)$	1.550
First floor column	$1.0(1.0)12.17$	12.17	0.023	2.0	0.046
Second floor girder	$1.33\left(\frac{19.17}{2}\right)1.04$	13.25	0.062	$\left(\frac{19.17}{4}\right)$	0.297
Continuous footing	$0.83(2.5)23.17$	48.4	0.225	0.41	0.092
Column footing	$2.67(1.67)0.83$	3.72	0.017	2.06	0.035
			3.484		5.172

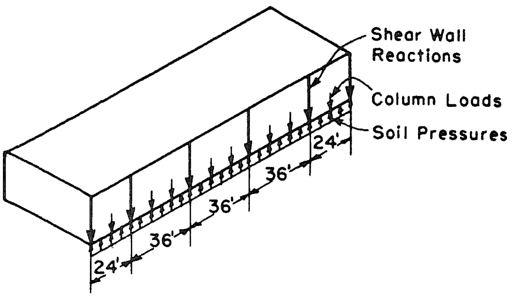
$\bar{x} = \frac{5.172}{3.484} = 1.485$ ft

Weight = 3.484(32.2) = 112.5 kips

* $\bar{x}$ is measured from exterior face of wall



b. <u>Determination of Maximum Vertical Force on Wall.</u> The critical value occurs at t = 0.0525. In order to determine the maximum shear and moment in the wall, a moment distribution will be performed using the loading at this time and the properties obtained in paragraph 9-29c.

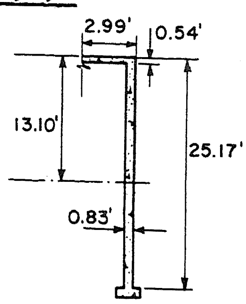


c. <u>Properties of Wall Section.</u>

$b' = 4t + b = 4(0.54) + 0.83 = 2.99$ ft

(ignore 2nd floor slab)

$$\bar{y}_{top} = \frac{2.16(0.54)0.27 + 0.83(25.17)12.58 + 2.5(0.83)25.58}{(2.16)0.54 + 0.83(25.17) + 2.5(0.83)}$$



$$\bar{y}_{top} = \frac{316.3}{24.15} = 13.10 \text{ ft}$$

$$A_w = (26.0)0.83 = 21.65 \text{ ft}^2$$

Table 9.43. Tabulation of Forces on Shear Wall from 24-ft Exterior Wall Section

a. Gravity load = 112.5 kips

b. Gravity load soil reaction (table 9.41) = 1.920 [(24.0)2.5 + (2.67)1.67] = 123.8 kips

c. Overturning soil reaction (table 9.41) = (pressure) [24.0(2.5) + (2.67)1.67] = 64.45 p

d. Inertial force = $mr\alpha$ (table 9.15) = 3.484(24.0 - 1.49) α = 78.5α

e. Blast load (table 9.27)

$\Sigma 1$ "b" edge of slab $\frac{1}{2}(2)$

2 "a" edges of slab

Load directly on wall and girder = $\left[(1.0)9.5 + 0.83(24.0)\right] \frac{\bar{P}_{roof}}{144} = 0.20\, \bar{P}_{roof}$

f. Soil reaction from blast load = (footing area)(pressure) = 64.45 p (table 9.41)

t (sec) \ Forces (kips)	a.	b.	c.	d.	e. 1 "b" edge	e. 2 "a" edges	e. $0.20\, \bar{P}_{roof}$	Σe.	f.	Net Back Wall (b + c + d + f - a - e)
0	112				0	0	0	0	0	
0.0025	112.5	123.8	3.2	98.2	2.48	2.24	0.1	4.82	5.2	113.1
0.005	112.5	123.8	14.8	198	6.51	6.06	0.3	12.87	21.9	233.1
0.0075	112.5	123.8	33.5	194	13.10	11.72	0.4	25.22	43.7	257.3
0.010	112.5	123.8	58.6	187	22.22	19.38	0.6	42.20	66.4	281.1
0.0125	112.5	123.8	90.8	172	32.80	28.24	0.6	61.64	94.0	306.5
0.015	112.5	123.8	123.8	162	43.20	37.16	0.8	81.16	121.0	341.1
0.0175	112.5	123.8	172	145	53.0	44.92	0.9	98.8	146.5	376.0
0.020	112.5	123.8	219	100	59.9	50.86	1.0	110.0	168.0	388.3
0.0225	112.5	123.8	272	76	64.2	54.70	1.1	120.0	180.0	419.3
0.025	112.5	123.8	327	48.7	66.7	57.10	1.27	125.1	189.0	450.9
0.0275	112.5	123.8	382	21.2	69.3	59.64	1.40	130.3	198.5	482.7
0.030	112.5	123.8	488	-57.2	73.0	63.0	1.53	137.5	208.0	512.6
0.0325	112.5	123.8	590	-88.0	79.6	68.6	1.66	149.8	232.0	606.1
0.035	112.5	123.8	686	-140	87.5	75.0	1.72	164.2	252.0	645.1
0.0375	112.5	123.8	782	-200	91.2	78.2	1.70	171.1	258.0	680.2
0.040	112.5	123.8	869	-254	93.2	80.0	1.69	174.89	262.0	714
0.0425	112.5	123.8	938	-304	94.4	80.6	1.66	176.7	264.0	733
0.045	112.5	123.8	990	-333	94.8	81.0	1.64	177.4	264.0	755
0.0475	112.5	123.8	1,030	-362	92.8	79.6	1.62	174.0	264	769
0.050	112.5	123.8	1,065	-386	88.2	75.8	1.61	165.6	248	773
0.0525	112.5	123.8	1,075	-397	83.5	71.6	1.60	156.7	238	780
0.055	112.5	123.8	1,075	-401	82.6	70.8	1.58	155.0	236	766
0.0575	112.5	123.8	1,060	-395	81.5	70.0	1.56	153.1	233	756
0.060	112.5	123.8	1,030	-381	81.0	69.4	1.55	152.0	231	738
0.0625	112.5	123.8	982	-357	79.8	68.4	1.53	149.7	227	713
0.065	112.5	123.8	927	-334	78.6	67.6	1.51	147.7	225	682
0.0675	112.5	123.8	856	-290						
0.070	112.5	123.8	774	-246						

The critical value occurs at t = 0.0525. In order to determine the maximum shear and moment in the wall, a moment distribution will be performed using the loading at this time and the properties obtained in paragraph 9-29c.

$I = (1/12)2.16(0.54)^3 + (0.54)2.16(12.83)^2 + (1/12)0.83(25.17)^3 + (0.83)25.17(0.52)^2 + (1/12)2.5(0.83)^3 + 2.08(11.66)^2 = 0.028 + 192.2 + 1{,}102.0 + 5.66 + 0.143 + 284$

$I = 1{,}583 \text{ ft}^4$

d. Constants for Moment Distribution - Center Spans (Par. 9-04).

$a_1 = 1, \quad a_2 = 0.5, \quad a_3 = 0.333$

$$S = \frac{I_o E}{L^2 A_o G} = \frac{(1{,}583)2.2}{(36.0)^2\ 21.65} = 0.1240$$

$$a_3' = a_1 S + a_3 = 0.1240 + 0.333 = 0.457$$

$$C_2 = \frac{a_2 - a_3'}{a_1 a_3' - (a_2)^2} = \frac{0.5 - 0.457}{0.457 - (0.5)^2} = 0.2075$$

$$C_1 = \left(\frac{a_3'}{a_2 - a_3'}\right) C_2 = \left(\frac{0.605}{0.5 - 0.457}\right) 0.2075 = 2.920$$

$$k = \frac{C_1 I_o}{4L} = \frac{(+2.920)1{,}583}{4(36.0)} = 32.1$$

$$C.O. = \frac{C_1}{C_1} = \frac{-0.2075}{2.920} = -0.0711$$

Modified k (symmetrical load) $= k(1 - C.O.) = 32.1\,[1 - (-0.0711)] = 34.4$

e. Constants for Moment Distribution - End Spans (Par. 9-04).

$$S = \frac{I_o E}{L^2 A_o G} = \frac{(1{,}583)2.2}{(24.0)^2\ 21.65} = 0.2780$$

$$a_3' = a_1 S + a_3 = 0.2780 + 0.333 = 0.61$$

$$C_2 = \frac{a_2 - a_3'}{a_1 a_3' - (a_2)^2} = \frac{0.50 - 0.611}{0.611 - (0.5)^2} = -0.308$$

$$C_1 = \left(\frac{a_3'}{a_2 - a_3'}\right) C_2 = \left(\frac{0.611}{0.5 - 0.611}\right)(-0.308) = 1.695$$

$$k = \frac{C_1 I_o}{4L} = \frac{(1.695)1{,}583}{4(24.0)} = 27.4$$

$$C.O. = \frac{C_2}{C_1} = \frac{-0.308}{1.695} = -0.182$$

$$\text{Modified } k \text{ (end span)} = k\left[1 - (C.O.)^2\right] = 27.4\left[1 - (-0.182)^2\right] = 28.3$$

f. <u>Load on 24-ft Span of Wall.</u>

Concentrated load at column:

Dead load (concrete volumes from table 9.31)

Roof slab $(0.54)12.0\left(\frac{19.17}{2}\right)$	= 62.0 cu ft
Second floor columns	= 10.0
Roof girder	= 12.2
Second floor slab $(0.46)12.0\left(\frac{19.17}{2}\right)$	= 52.8
First floor columns	= 12.17
Second floor girder	= 13.25
	152.4 cu ft

$$152.4\left(\frac{150}{1{,}000}\right) = 22.9 \text{ kips}$$

Blast load on 1 "b" edge ($t = 0.0525$ sec)(table 9.43) = 83.5 kips

Blast load directly on girder ($t = 0.0525$)

$$= \frac{(1.0)9.5}{144}(\bar{P}_{roof}) = 0.066(8.0) = 0.5 \text{ kips}$$

Total load on column 106.9 kips

Assume that the attached column footing soil pressure is a concentrated load. Subtracting this from the total column load above equals net concentrated load.

Area of attached footing = 2.67(1.67) = 4.40 sq ft

Net soil pressure at $t = 0.0525$ is 22.31 kips/ft^2 (table 9.41)

Upward pressure under column = 4.40(22.31) = 98.0 kips

Net concentrated load = (106.9 - 98.0) = 8.9 kips

Uniformly distributed load:

Soil pressure = 22.31(2.50)24.0	=	1,338.0 kips ↑
Gravity load (112.5 - 22.9)	=	89.6 ↓
Blast force (2 "a" edges)	=	71.6 ↓
Inertial force (table 9.43)	=	397.0 ↓
Net uniform load	=	780.0 kips ↑

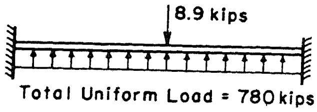
g. Analysis of Deep Beam Using Moment Distribution.

Fixed end moments for 24-ft span:

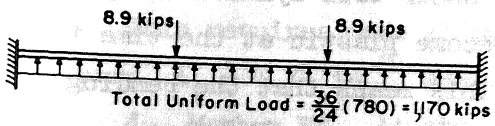
Unif FEM $= \frac{1}{12}$ WL $= \frac{1}{12}(780)24 = 1{,}560$ ft-kips

Conc FEM $= \frac{PL}{8} = -\frac{8.9(24)}{8} = -26.7$ ft-kips

Net FEM $= +1{,}533$ ft-kips



Fixed end moments for 36-ft span:



Unif FEM $= \frac{1}{12}$ WL $= \frac{1}{12}(1{,}170)36$

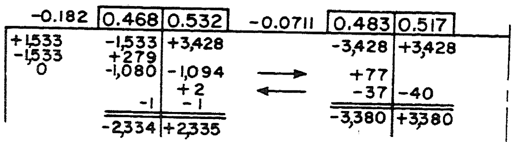
$= 3{,}500$ ft-kips

Conc FEM $= \frac{P(ab^2 + a^2b)}{L^2} = \frac{Pab(a+b)}{L^2} = \frac{Pab}{L} = -\frac{8.9(12)24}{36} = 71.4$ ft-kips

Net FEM = 3,428.6 ft-kips

Moment distribution:

(See sketch at right.)



h. Shear Strength.

Rear wall:

max V = 605.2 kips

$v = \frac{605.2(1{,}000)}{0.83(26.0)144} = 194$ psi

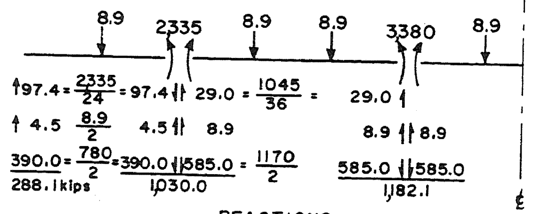
$194 < 390 = 0.10\ f'_{dc}$ $\therefore$ OK

Shear wall:

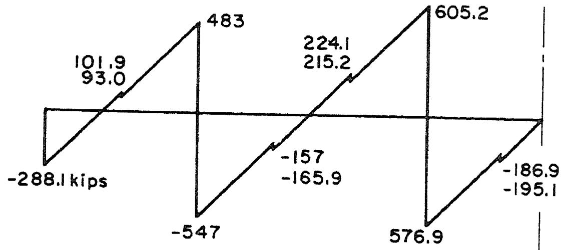
Maximum reaction = 1,182.1 kips

$v = \frac{1{,}182.1(1{,}000)}{0.83(26.0)144} = 378$ psi

$378 < 390 = 0.10\ f'_{dc}$



REACTIONS



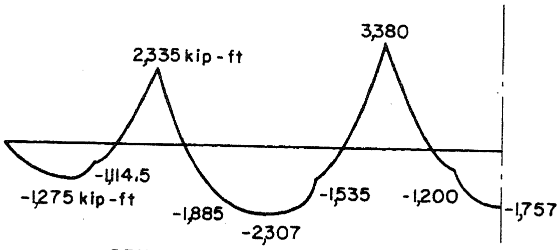
SHEAR DIAGRAM

i. Steel Required for Bending.

Maximum moment = 3,380 ft-kips

$A_s = \frac{M}{f_{dy}d} = \frac{3{,}380}{52(25.5)} = 2.55\ \text{in.}^2$

(eq 4.57)



BENDING MOMENT DIAGRAM

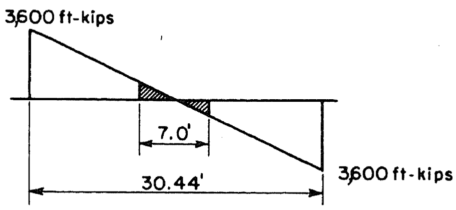
Bond requirement

$$\Sigma o = \frac{V}{ujd} = \frac{605.2(1,000)}{450\left(\frac{7}{8}\right)25.5(12)} = 5.02 \text{ in.}$$

Use 2 #10 bars at top and bottom, $A_s = 2.54$, $\Sigma o = 9.0$ in. The steel requirement for other sections of the wall can be determined in proportion to the moment and checked for bond as indicated in the shear and moment diagrams above.

9-30 <u>FINAL DESIGN OF SHEAR WALLS.</u> During the shear wall dynamic analysis (par. 9-26), certain members were assumed to become plastic at the time the shear stress in the member exceeded 600 psi. This means that the bending moment cannot increase above the value it has attained when the shear stress reached the limiting value of 600 psi. This value of limiting bending is computed for the various shear wall members in paragraph 9-26d.

a. <u>Investigation of Moment and Shear at Opening - Interior Shear Walls.</u> The moment acting on the lintel and footing members will be considered to be end moments as obtained from the limiting shear values.



b. <u>Member AB.</u> At V = 236 kips, the section was permitted to become plastic. Evaluating moment for this shear,

$$M = \frac{236(30.44)}{2} = 3,600 \text{ ft-kips.}$$

$$M = \frac{7.0}{30.44}(3,600) = 825 \text{ ft-kips (at face of opening)}$$

$$A_s = \frac{M}{f_{dy}d} = \frac{825}{52(3.5)} = 4.54 \text{ in.}^2, \text{ use 6 \#8 bars, } A_s = 4.74 \text{ in.}^2$$

$$p = \frac{4.74}{10(3.5)12} = 0.01130$$

Shear investigation. For a shear intensity of 600 psi, stirrups are essential.

$$v = 0.04 f'_c + 5,000 p + rf_y \quad \text{(eq 4.24)}$$

$$600 = 0.04(3,000) + 5,000(0.0113) + r(40,000)$$

$$r = \frac{600 - 176.5}{40,000} = 0.0106$$

Web reinforcement $= r(10)12 = 0.0106(10)12 = 1.270$ in.^{2}/ft

Use U stirrups #5 at 6 in., $A_s = 1.24$ in.^{2}

c. Member CD. For v = 600-psi allowable stress, V = vbjd = (600)10(7/8)45 = 236 kips. Evaluating moment for this shear - M = $\frac{236(31.37)}{2}$ = 3,700 ft-kips

$$\text{Moment at opening} = \frac{7.0}{31.37}(3{,}700) = 826 \text{ ft-kips}$$

$$A_s = \frac{M}{f_{dy}d} = \frac{826}{52(3.5)} = 4.54 \text{ in.}^2$$

This requires same reinforcement as member AB in b above, namely 6 #8 bars, A_s = 4.74 in.^{2}, and U stirrups #5 at 6 in., A_s = 1.24 in.^{2}.

d. Member EF. At the time that the lower shear wall became plastic the member EF was not permitted to take further moment because of the assumption of plastic hinges at the end of member AB. This is evaluated in table 9.40. M_{EF} = 5,730 ft-kips.

Design moment at face of opening

$$M = \frac{7.0}{31.37}(5{,}730) = 1{,}280 \text{ ft-kips}$$

$$A_s = \frac{M}{f_{dy}d} = \frac{1{,}280}{52(2.50)} = 9.85 \text{ in.}^2$$

Use 10 #9 bars in one row, A_s = 10.0 in.^{2}

$$p = \frac{9.85}{(30)30} = 0.0109$$

$$\text{Shear throughout member} = \frac{2 \text{ moment}}{\text{span}} = \frac{2(5{,}730)}{31.37} = 366 \text{ kips}$$

$$\text{Shear intensity } v = \frac{V}{bjd} = \frac{366(1{,}000)}{30\left(\frac{7}{8}\right)33} = 464 \text{ psi}$$

$$\text{Allowable shear stress} = 0.04f'_c + 5{,}000\,p + rf_y \quad (\text{eq } 4.24)$$

$$464 = 0.04(3{,}000) + (5{,}000)0.0109 + r(40{,}000)$$

$$r = \frac{464 - 120 - 54.5}{40{,}000} = 0.00725$$

Area of steel required for shear reinforcement per foot of beam = 0.00725(30)12 = 2.61 in.^{2}

Use U stirrups #7 at $5\frac{1}{2}$ in., A_s = 2.62 in.^{2}

e. Investigation of Moment and Shear of Footing - End Shear Wall. At the time that the lower shear wall became plastic, the member EF was not permitted to take further increase in moment. This occurred at time $t = 0.0300$ sec. From table 9.40, $M_{EF} = 2{,}380$ ft-kips. Design moment at face of opening

$$M = \frac{7.0(2{,}380)}{29.76} = 560 \text{ ft-kips}$$

$$A_s = \frac{M}{f_{dy}d} = \frac{560}{52(2.50)} = 4.32 \text{ in.}^2$$

Use 10 #6 bars in one row, $A_s = 4.40$ in.^{2}, $p = \frac{4.40}{30(30)} = 0.00488$

$$\text{Shear throughout member} = \frac{2(\text{moment})}{\text{span}} = \frac{2(2{,}380)}{29.76} = 160 \text{ kips}$$

$$\text{Shear intensity } v = \frac{V}{bjd} = \frac{160(1{,}000)}{30\left(\frac{7}{8}\right)33} = 184 \text{ psi}$$

Allowable shear stress $= 0.04f'_c + 5{,}000\,p + rf_y$ (eq 4.24)

$$184 = 0.04(3{,}000) + 5{,}000(0.00488) + r(40{,}000)$$

$$r = \frac{184 - 120 - 24}{40{,}000} = 0.00090$$

Area of steel required for shear reinforcement per foot of beam $= 0.0009(30)12 = 0.324$ in.^{2}

Use U stirrups #3 at 8 in., $A_s = 0.34$ in.^{2}

f. Shear Panel Reinforcement. The shear walls are divided by the corridor openings into two shear wall panels joined together at the openings. The shear wall reinforcement is calculated using the width and vertical span of the individual shear wall panels rather than the entire shear wall. In the shear wall analysis the maximum cracking resistances of the shear panels were developed, hence the steel reinforcement will be proportioned to provide an ultimate strength in the panels equal to the resistances at which the panels crack.

g. Interior Shear Walls (Second Story).

Vertical span of shear panels = 11.6 ft (par. 9-25j)

Width of shear panels = 20.5 ft (par. 9-25e)

Length center to center of vertical edge steel = 20.5 - 2 = 18.5 ft = L

Maximum total shear wall resistance = 1,200 kips(table 9.40)

Maximum shear per panel = 1,200/2 = 600 kips = R_{du}

Minimum panel steel will be used ($p = 0.0025$) $A_s = 0.0025(10)12$

$= 0.30$ in.^{2}/ft. Use #3 at 9 in.

Referring to equation 4.55:

$P = f_{dy}\ pt(H + L) = 52(0.0025)10(11.6 + 18.5) = 390$ kips

$P/R_{du} = 390/600 = 0.650$ (single story)

From figure 4.36, $P/C = 0.87$; $C/R_{du} = 0.74$

Required $C = 0.74(390) = 288$ kips

$C = A_s\ f'_{dy}\left[15 + 1.9(L/H)^2\right] = 288 = A_s(3.9)\left[15 + 1.9(18.5/11.6)^2\right]$

$A_s = \dfrac{288}{3.9(19.82)} = 3.73$ in.^{2}

Use 4 #9 bars as edge steel for shear panels, $A_s = 4.0$ in.^{2}

h. Interior Shear Walls (First Story).

Vertical span of shear panels = 11.6 ft (par. 9-25g)

Width of shear panels = 20.5 ft (par. 9-251)

Length center to center of vertical edge steel = 20.5 - 2

= 18.5 ft = L

Maximum shear per panel = 1,020 kips = R_{du} (table 9.40)

Minimum panel steel will be used ($p = 0.0025$), $A_s = 0.0025(10)12$

$= 0.30$ in.^{2}/ft. Use #3 at 9 in.

Referring to equation (4.55):

$P = f_{dy}\ pt(H + L) = 52(0.0025)10(11.6 + 18.5)12 = 471$ kips

$P/R_{du} = 471/1{,}020 = 0.462$

From figure 4.36, $P/C = 0.78$; $C_d/R_{du} = 0.59$

Required $C = 0.59(1{,}020) = 602$ kips

$C = A_s\ f'_{dc}\left[15 + 1.9(L/H)^2\right] = 602 = A_s(3.9)\left[15 + 1.9(18.5/11.6)^2\right]$

$A_s = \dfrac{602}{3.9(19.84)} = 7.78$ in.^{2}

Use 8 #9 bars as edge steel for shear panels, $A_s = 8.0$ in.^{2}

i. End Shear Walls (Second Story).

Vertical span of shear panels = 11.5 ft = H

Width of shear panels = 48.0 ft

Length center to center of vertical edge steel = 48.0 - 2.0

= 46.0 = L

Maximum shear per panel = 1,564 kips (table 9.40) = R_{du}

Minimum panel steel will be used ($p = 0.0025$), $A_s = 0.0025(10)12 = 0.30$ in.^{2}/ft. Use #3 at 9 in.

Referring to equation (4.55):

$P = f_{dy}\ pt(H + L) = 52(0.0025)10(11.5 + 46.0)12 = 896$

$P/R_{du} = 896/1{,}564 = 0.572$ (single-story wall)

From figure 4.36, $P/C = 0.710$; $C/R_{du} = 0.79$

Required $C = 0.79(1{,}564) = 1{,}235$ kips

$C = A_s\ f'_{dc}\left[15 + 1.9(L/H)^2\right] = 1{,}235 = A_s(3.9)\left[15 + 1.9(46.0/11.5)^2\right]$

$A_s = \dfrac{1{,}235}{3.9(45.2)} = 7.00$ in.^{2}

Use 6 #10 bars as edge steel for shear wall, $A_s = 7.62$ in.^{2}

j. <u>End Shear Walls (First Story).</u>

Vertical span of shear panels = 9.0 ft

Width of shear panels = 21.0 ft

Length center to center of vertical edge steel (21.0 - 2.0) = 19.0 = L

Maximum shear per panel = 1,030 kips = R_{du} (table 9.40)

Minimum panel steel will be used ($p = 0.0025$), $A_s = 0.0025(10)12 = 0.30$ in.^{2}/ft. Use #3 at 9 in.

Referring to equation (4.55)

$P = f_{dy}\ pt(H + L) = 52(0.0025)10(9.0 + 19.0) = 364$ kips

$P/R_{du} = 364/1{,}030 = 0.353$ (for multi-story wall)

From figure 4.36, $P/C = 0.51$; $C/R_{du} = 0.69$

Required $C = 0.69(1{,}030) = 711$ kips

$C = A_s\ f'_{dy}\left[15 + 1.9(L/H)^2\right] = 711 = A_s(3.9)\left[15 + 1.9(19.0/9.0)^2\right]$

$A_s = \dfrac{711}{3.9(23.48)} = 7.77$ in.^{2}

Use 8 #9 bars as edge steel for shear panels, $A_s = 8.0$ in.^{2}

9-31 <u>DESIGN SUMMARY.</u> Figure 9.33 shows the locations of the various elements that were designed in this example. Sections of the various elements are shown in figures 9.34 through 9.41 to indicate the final design. References are given on each section to the paragraphs where each design is first presented.

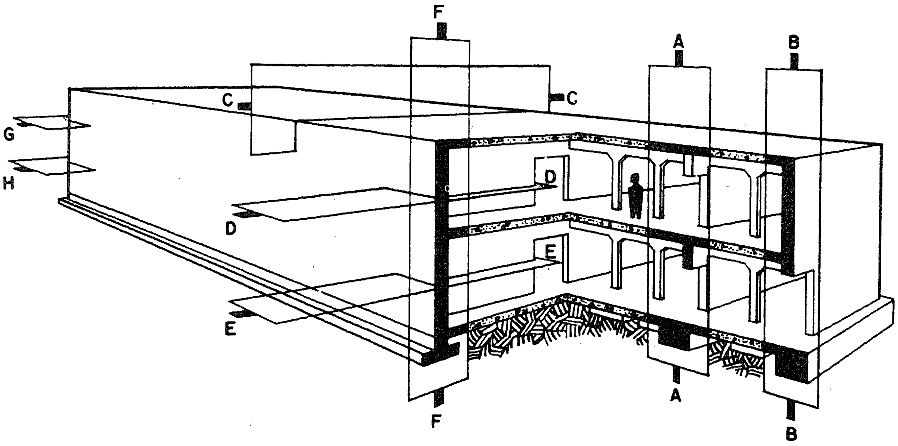


Figure 9.33. Locations of design section

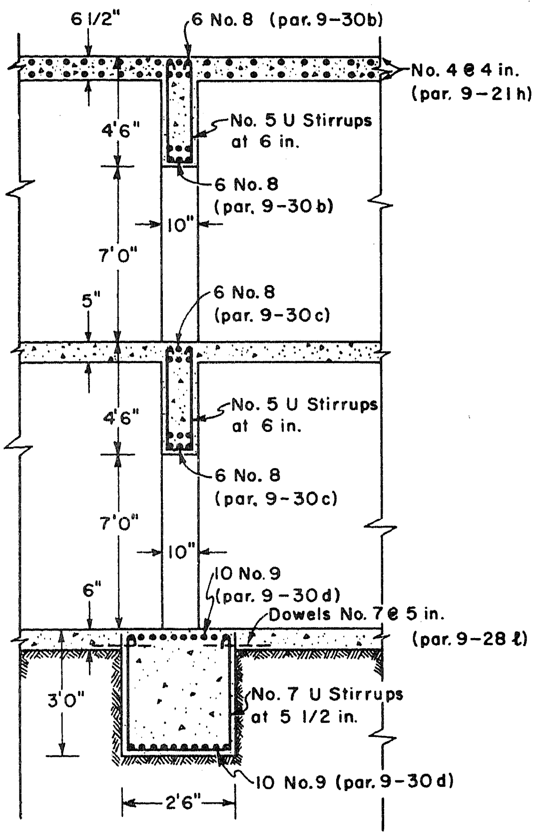


Figure 9.34. Section A-A of figure 9.33

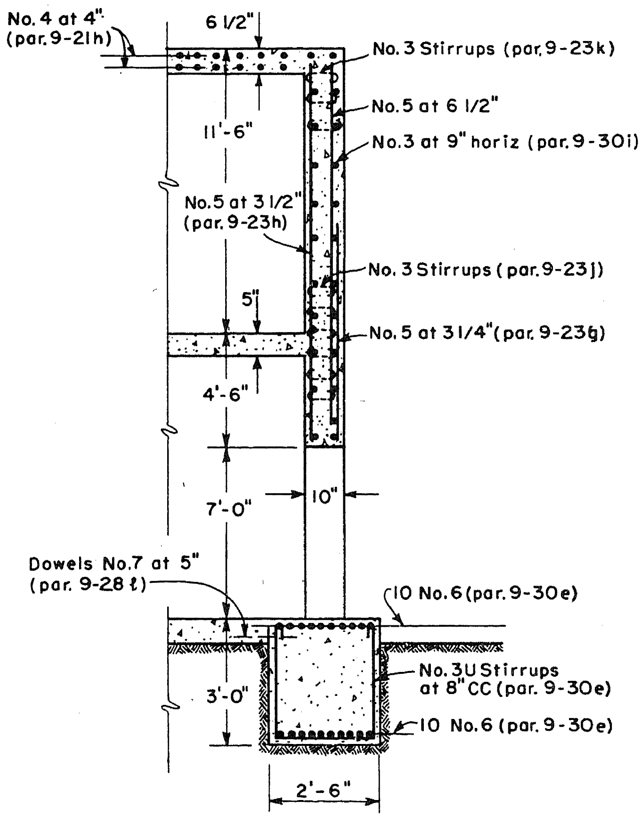


Figure 9.35. Section B-B of figure 9.33

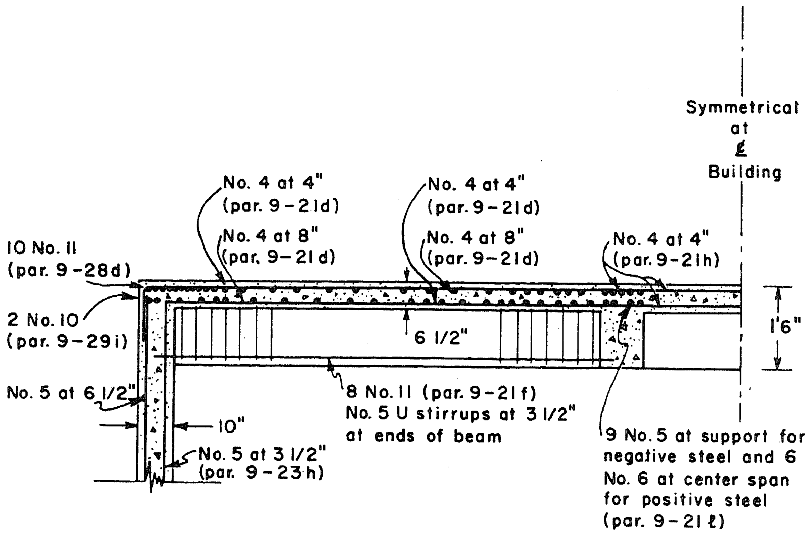


Figure 9.36. Section C-C of figure 9.33

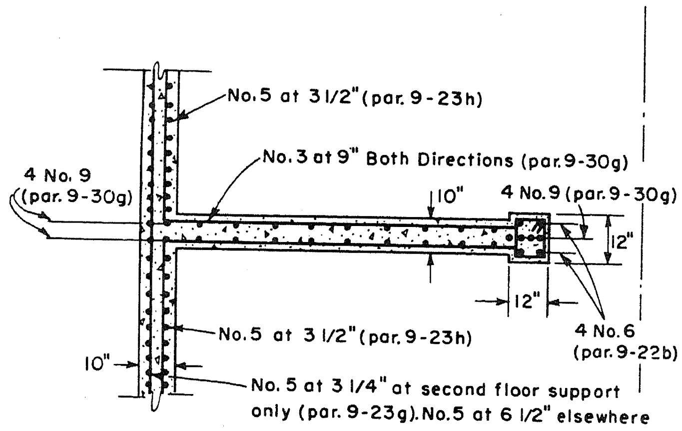


Figure 9.37. Section D-D of figure 9.33

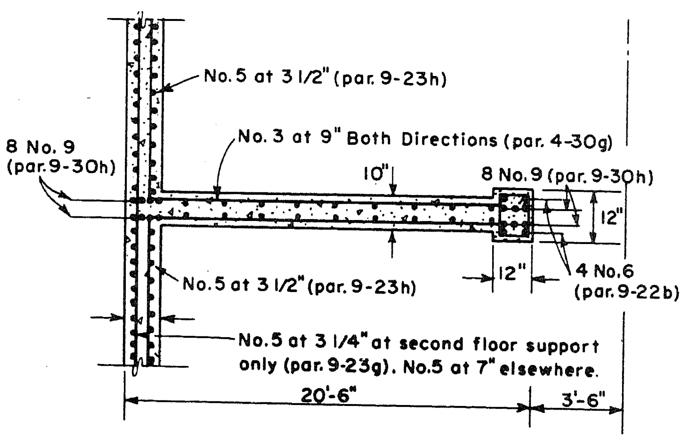


Figure 9.38. Section E-E of figure 9.33

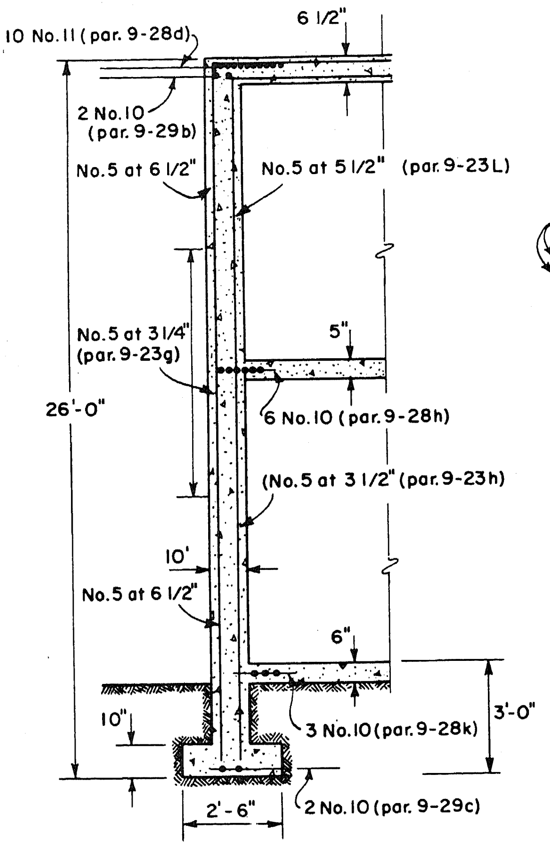


Figure 9.39. Section F-F of figure 9.33

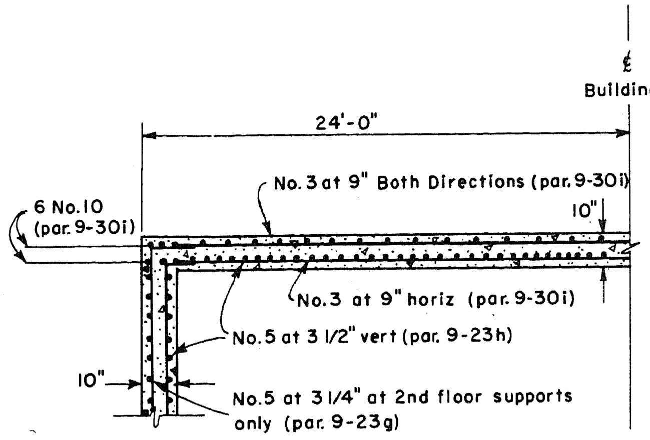


Figure 9.40. Section G-G of figure 9.33

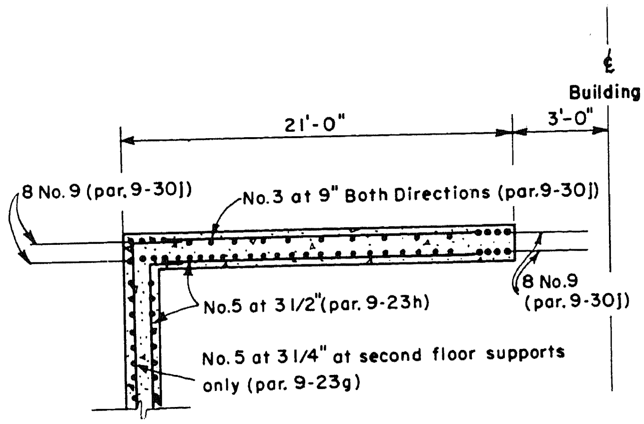


Figure 9.41. Section H-H of figure 9.33

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